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OF GENERAL INTEREST

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The papers "The Water Supply Tunnels of the Boston Metropolitan District" by Karl R. Kennison, published in the January, 1949 JOURNAL, and "Pollution of the Androscoggin River by Industrial Wastes and Control Measures Thereof" by E. Sherman Chase, published in the July, 1949 JOURNAL were presented in a joint meeting of the Sanitary Division of the American Society of Civil Engineers and the Boston Society of Civil Engineers, held in Boston on October 14, 1948.

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BOSTON MAIN DRAINAGE WORKS PROPOSED CALF PASTURE SEWAGE TREATMENT PLANT

By John F. Flaherty, Member*

(Presented at a joint meeting of the Boston Society of Civil Engineers and the Sanitary Section, B.S.C.E., held on May 18, 1949.)

INTRODUCTION

The City of Boston, through its Public Works Department, has had prepared plans and specifications for the construction of a treatment plant to treat the sewage from the Boston Main Drainage district. The purpose of this paper is to present a brief description of the development and extent of the Main Drainage system and an outline of the studies and reports which preceded the preparation of plans for the treatment plant.

The Boston Main Drainage System

There are three major sewerage systems in Metropolitan Boston: namely, the North Metropolitan System, the South Metropolitan System, and the Boston Main Drainage System, see Fig. 1. The North Metropolitan Sewerage District, established by the Legislature in 1889, serves the East Boston and Charlestown districts of Boston and several communities to the north of Boston. The South Metropolitan Sewerage District, established in 1899, disposes of the sewage from communities to the west and south of Boston and from the Brighton and Hyde Park districts and parts of the West Roxbury, Dorchester, and Roxbury districts of Boston. These two systems are under the jurisdiction of the Metropolitan District Commission. The Boston

*Senior Civil Engineer, Sewer Division, City of Boston.

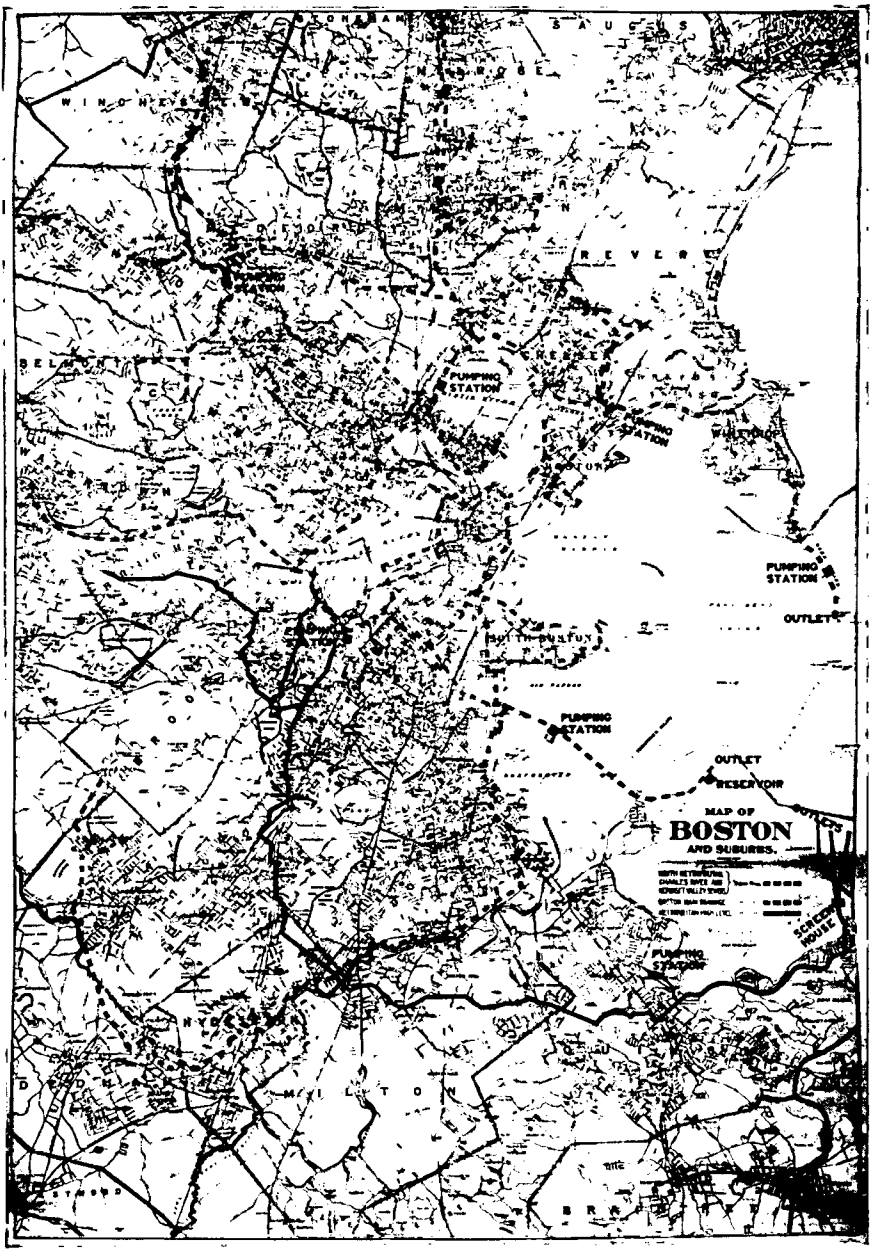

FIG. 1.—BOSTON MAIN DRAINAGE METROPOLITAN SEWERS.

Main Drainage System was established in 1876 and at present receives the sewage from Boston Proper, South Boston and parts of the Back Bay, Roxbury, Dorchester, West Roxbury, Milton and Squantum. This system is under the jurisdiction of the City of Boston.

By 1873, there were about 125 miles of sewers in Boston, having some 70 separate outlets along the harbor shores and its tidal estuaries. This system of sewerage which was being constantly extended through low districts on very flat grades, without a definite, comprehensive plan, resulted in pollution and offensive odors, and led to a demand for a sewage disposal system to correct these evils.

The Boston Main Drainage system was designed to intercept sewage from the numerous outlets and effect its disposal at a remote location. The intercepters were designed on the combined system to carry the sewage from an area of 20 square miles, having a population of 800,000, contributing an average per capita discharge of 75 g.p.d., or a total discharge of 139 c.f.s. Provision for storm water was made by adding 100 c.f.s., which represents about one-fourth of an inch rainfall in 24 hours, but it was intended to admit little storm flow from regions not subject to flooding, reserving the capacity to relieve low-lying districts. When the capacity of the intercepters was reached, overflow was to take place through the existing outlets; tide gates and, in many cases, regulators being installed at these overflows.

The Main Drainage system at present serves an area of 11,500 acres, a resident population of 450,000 persons, and a transient population of over 825,000 persons from whom an equivalent sewage discharge equal to a 50,000 resident population is estimated. It consists of over 27 miles of intercepting sewers, chiefly of brick masonry, in size up to 10′-6″ in diameter, which flow by gravity to the Calf Pasture pumping station. The disposal works comprise elevator cage screens, a pumping station and deposit sewers at Calf Pasture, a tunnel under Dorchester Bay to Squantum, an outfall sewer from Squantum to Moon Island, and storage basins at Moon Island (Fig. 2) which are discharged on the ebb tide.

The two controlling elevations imposed on the new plant design by the existing disposal works are the elevations of the structures immediately upstream and downstream of the proposed plant. Upstream the grades of the intercepters were governed by the elevations of the sewers to be intercepted with a result that the main

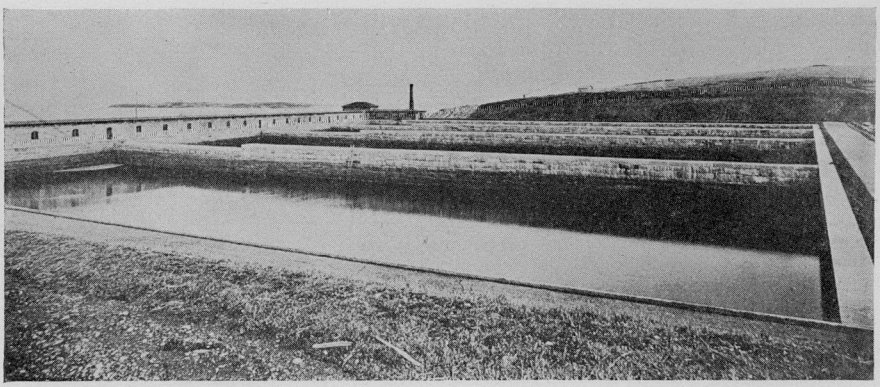

FIG. 2.—STORAGE RESERVOIR AT MOON ISLAND.

intercepter at its terminus at Calf Pasture has an invert elevation of —13.65, thus its crown is over 3 ft. below mean low water. No discharge into the harbor at this point is possible without a head upstream sufficiently high to discharge against the tidal elevation. Such head would cause extensive flooding in the low lying areas and such low velocities as to result in deposits in the sewers. Therefore no by-pass could be provided and all quantities to be disposed of must be lifted by pumping. With the ground surface at elevation 17, cuts in excess of 30 ft. will be required for the screen and grit structures and the pumping station substructure. Downstream the water line in the deposit sewers depends on the head required to pass the flow through the Dorchester Bay tunnel. The water line elevation varies with the quantities of pumpage and storage, averaging about elevation +27, imposing a static lift on the pumps of about 35 ft. and establishing the limits of the water line elevations of the proposed aeration channels and settling tanks.

Another factor which had an important effect upon the plant design is the capacity of the Dorchester Bay tunnel (Fig. 3) and the surges that occur at the West and East shafts. This tunnel is an inverted siphon brick lined, 7′-6″ in diameter and 7,160 ft. long. Its lowest point is 143 feet below mean low water. Its present capacity is about 160 MGD. A comparison with the main intercepter capacity of 210 MGD and the outfall sewer capacity of 272 MGD shows the tunnel to be a bottleneck in the system. Observations of surging at the tunnel shafts show constantly recurring surges of about 5 ft. on a 4-minute cycle, generally during periods of low

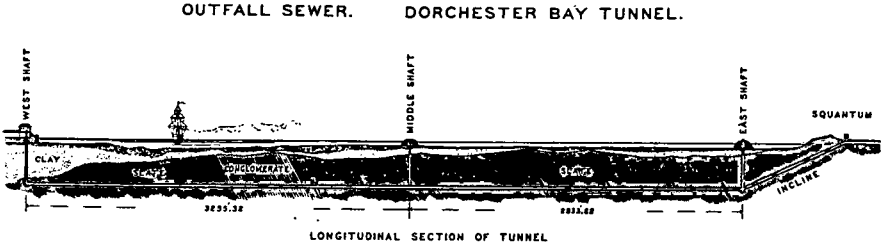

FIG. 3.

pumping rates. Individual surges of a magnitude of over 19 ft. at the East shaft and up to 11 ft. at the West shaft are eccentric in occurrence. Although several theories have been suggested, the cause of the surging has not yet been definitely established.

TREATMENT PLANT STUDIES

Since the year 1900, the discharge of sewage into Boston Harbor has been investigated a number of times by direction of the Legislature, the most notable being in 1900, 1917, 1929, 1935,* and 1938. The reports dealt with the changes required in the systems of sewerage and sewage disposal to prevent nuisances or to remove objectionable conditions. The Special Commission of 1935 also considered the sanitary conditions of the harbor waters and their suitability for bathing purposes, the advisability of extending the outlets to points remote from shore and the possible treatment of sewage to improve the harbor waters from a sanitary standpoint.

The Commission of 1938 contracted for the engineering services of Metcalf and Eddy, of Boston, and Greeley and Hansen, of Chicago. The engineers were given several problems to solve for the Commission, one of them being to outline a proposed method of sewage treatment for the Boston Main Drainage District.

The Engineers Report contained in House No. 2465* of 1939, recommended that a $14,700,000 program be formulated for the construction of storm water overflow conduits, relief sewers and similar works along the inland tributaries to the harbor and a $10,000,000 construction program be undertaken for the treatment of sewage from the North and South Metropolitan Sewerage Districts and the Boston Main Drainage District.

*See House No. 1600, 1936.

*See also: Discussion of Problems of Sewerage and Sewage Disposal in Metropolitan Boston, Eddy and Fales. Jour. B.S.C.E., 1940, p. 79, and Pollution of Boston Harbor, S. A. Greeley, Jour. B.S.C.E., 1940, p. 102.

The degree of treatment recommended for the three districts was the same: namely, (1) to remove floating solids which are unsightly on the harbor waters and may be carried to the shores by wind and tide; (2) to remove grease which causes extensive sleek areas on the harbor waters; (3) to reduce the amount of suspended solids capable of forming sludge deposits; and (4) to prevent excessive bacterial pollution of harbor and shore waters during the recreational season. These objectives were to be accomplished through preliminary treatment by means of racks and grit chambers and through primary treatment in aerochlorination tanks, plain sedimentation tanks and chlorination of the effluent during the recreational season. Raw sludge was to be disposed of by barging to sea.

The Engineers Report to the 1938 Commission has been the basis for the plans that have since been developed both by the City of Boston and the Metropolitan District Commission. While the original plans have been changed considerably in regards to the layout of the Main Drainage treatment plant, the recommendations for the degree and method of treatment, except for sludge disposal, have remained substantially unchanged.

Additional Treatment Plant Studies

The engineers of the Boston Sewer Division, after a review of the reports previously referred to, proceeded to make further studies and prepare preliminary plans for a treatment plant in more detail than those outlined previously.

Five major factors were investigated: namely, (1) the existing rates of flow and their duration; (2) the possibility of utilizing some of the existing structures for containing part of the treatment plant processes; (3) the possibility of eliminating the necessity of a low lift pumping station at Moon Island; (4) the effect of placing the Dorchester Bay tunnel under additional head to increase its capacity, and (5) the effect any changes in plant layout would have on construction and operating costs.

In September of 1945, the City of Boston contracted for the services of the Charles A. Maguire and Associates, Engineers, who in turn engaged Elson T. Killam as Consultant, to make preliminary studies, plans, and estimates, and to prepare final designs, detailed working drawings and specifications for a sewage treatment plant. The plans for the treatment works have been completed, and the papers that follow will describe the plant layout and design.

PROPOSED CALF PASTURE SEWAGE DISPOSAL PLANT

By Eric Reeves*

A bird's-eye view of the main units which comprise the proposed Calf Pasture Sewage Disposal Plant is shown in Fig. 4. This

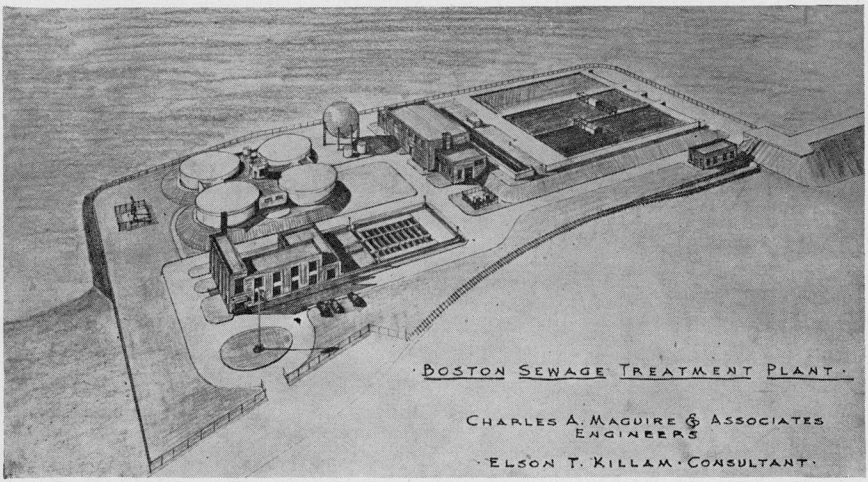

Fig. 4.

paper deals briefly with the architectural, structural and foundation features of the various units.

The perspective view shows the Administration Building facing the main gate beyond the traffic circle and the flag pole. Underneath the Administration Building will be the screen house. Adjoining the screen house and extending to the right will be the open grit chamber. Around the grit chamber there will be an enclosed outdoor transformer station. Attached to the larger building, the heart of the plant—the pumping station will be the machine shop. On the far side of the pumping station the gas compressor house will be located. Behind the pumping station there will be the hair-pin-shaped aeration channel connecting through the gate chamber to the three settling basins—a fourth gate being provided to serve a future

*Engineer, Charles A. Maguire Associates, Consulting Engineers.

fourth tank. The settling tanks will be equipped with two Mieder machines for sludge scraping. The two existing south and north deposit sewers to which this plant will be connected will be on the extreme right on top of the embankment. The former sewer will discharge all normal flow into the existing tunnel to Moon Island, while the latter will discharge the excess over the tunnel capacity into Dorchester Bay. At the end of the tracks the Chlorination House will be located with facilities to chlorinate automatically all bypass effluent at all times, and the entire effluent during the bathing season.

The main purpose of the railroad spur is to bring in tank chlorine.

There will be four sludge digestion tanks—two primary, two secondary. Between the tanks, shaped like a symmetrical maltese cross will be the two-story digester control house. A hortonsphere for storage of compressed gas will be located to the right of one digestion tank and fronting it will be two low pressure gas receivers. Under the area fronting the other tank fuel oil storage tanks will be buried. On the extreme left, enclosed and isolated, the two waste gas burners will be placed.

The plant will occupy an area 812′ long by 415′ wide—approximately eight acres. All units will be easily accessible by 20′ macadam service roads. For fire protection, each unit can be completely covered by at least two fire hydrants. For the protection of the workmen, all open channels will be flood-lighted at night in addition to normal street lighting. For juvenile protection, the entire plant will be surrounded by a 6′ chain link fence and all roof ladders will be accessible only from the inside of the buildings.

Based on present-day prices, the completed plant, including equipment, will probably cost in the neighborhood of 8 million dollars.

Screen, Grit and Administration Building. The Administration Building itself will be of reinforced concrete frame and 12″ exterior brick walls with 4″ face brick with decorative aluminum spandrels between the windows.

The subterranean portions of this unit—the combined screen room and grit chamber—will be 244′ long x 72′ wide and 30′ below grade. Under normal operating conditions and with normal tides, there would be a net foundation pressure of less than 100 lbs. per sq. ft.; but it is possible although not probable, to have an abnormally high tide combined with empty chambers, in which case there would be a tendency for uplift. To overcome this buoyancy, the dead

weight of the administration building was added to the screen room and wood piles to the grit chamber. Piles will be capable of sustaining an uplift of 10 tons.

The first floor will be "U" shaped. The right-hand side of the U will consist of a three-car garage and a small workshop and storage room. Adjacent to this the workmen's locker room, wash room and lunch room will be located. The bottom of the U will consist of the main entrance, the operator's room, first-aid room and stairway to the screen chamber below. The left-hand side of the U will be an equipment room, housing principally the three-way controls for each of the two main sewage inlet gates, and the gas fired boiler. This room will be serviced by a ten-ton bridge crane principally for the gate control equipment but will also be used to service the screen room below.

The second floor, from left to right, will consist of the chemist's private office, the specimen room, a fully equipped laboratory and the office of the plant superintendent. Two large exhaust ducts with two 9,600 c.f.m. fans can provide six air changes per hour for the screen chambers if required; and inaddition, on the extreme left two exhaust ducts will operate continuously under pressure from the two inlet conduits to draw off any sewage gases which could otherwise enter the screen chamber.

Pump House. The front elevation of the pump house structure is shown in Fig. 5. The compressor house is shown on the left and

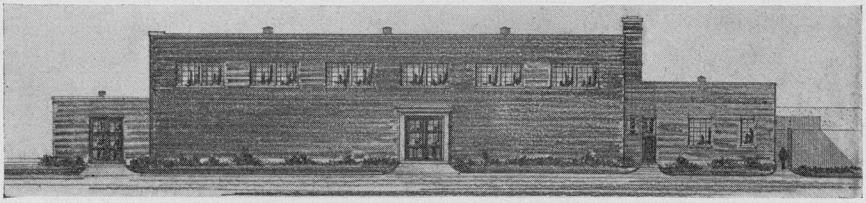

FIG. 5.—PUMPING STATION.

the machine shop on the right. The superstructure will be a one-story steel framed building with 12″ brick walls including 4″ of face brick. The face brick will be salmon colored iron spotted. The roof will be 3½″ precast concrete slabs on steel purlins. The pump house proper will be approximately 130′ long by 62′ wide and will be serviced by a 60′ bridge crane. The substructure will be a two-

story basement all reinforced concrete, extending approximately 45′ below grade.

On the left of the main entrance will be three dual fuel engines, while to the right the electric pumps, aeration blower, main electric switch board and gauge panel will be located. Four hatchways will be provided to service the equipment room and the pump room below. Adjacent to the far wall there are five openings for the pump discharge cones. Access and escape stairs will be located at either end of the building. Fully equipped machine shop, electric shop and storage bins, workmen's entrance and lockers will be provided.

The pump house will be founded in the soft blue clay, as indicated by the adjacent boring. Fortunately, the weight of the excavated material will approximate the weight of the building so that when completed there will be a net increase in pressure on the clay of about 200 lbs. Incidentally, the existing pump house was designed to float on a wooden raft. The sand ballast cell on the right will serve to bring the center of gravity in vertical alignment with the center of buoyancy giving approximately even distribution of pressure.

Sludge Digestors

A front elevation of the sludge digestion unit, as viewed from the pumping station, is shown in Fig. 6. Four concrete tanks will

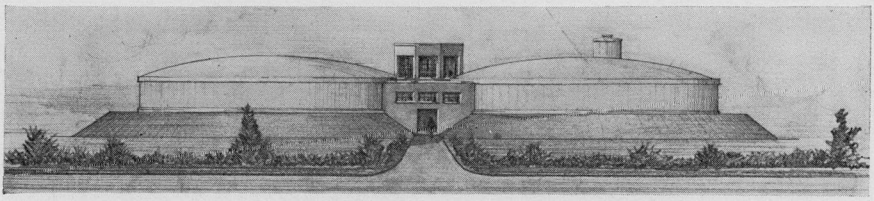

Fig. 6.—Sludge Digestor House.

enclose a steel framed concrete two-story control house. The unit will be symmetrical except for the small scum hopper on the roof at the right. The cupola on the roof will provide light and ventilation, access to the roof for sludge sampling and quick vent in case of a gas explosion. The earth berms will serve as insulation to maintain the temperature of the tanks. An underground pipe and service tunnel will connect this unit with the pumping station.

The digestion tanks will be 75′ in diameter by 26′-6″ deep at the

outside walls and 8′-10½″ deeper at the center. The maximum capacity will be almost one million gallons. Structurally the tanks will be reinforced concrete cyinders, the shell thickness varying with the depth and consequent pressure. The bottoms will be cone shaped. The sides will be joined to the bottoms by a continuous horizontal expansion joint to permit unrestrained expansion and contraction of the walls. Because of this construction, the adjoining floors of the control house will have a 1″ cork expansion joint and all supporting beams which rest on the tank walls will be able to move freely on lubrite expansion plates. The piles under the digestion tanks, as in the grit chamber, provide an insurance against buoyancy in the event of an extremely high tide and empty tanks.

Settling Basins

The settling tanks will be approximately 300′ long, 245′ wide by 18′ deep with a 2′ free board. The bottom of the tanks will be approximately at grade. This means that on the seaward side, there will be a new fill of 16′ to which will be added the weight of the tanks themselves. This additional load of two tons per square foot will be transferred by piles through the fill to a 20′ sand layer below and from the sand layer into the underlying Boston soft blue clay. Incidentally, here is the deepest layer of blue clay yet disclosed in the Boston area—175 feet.

With this load and with this depth of clay based on compression tests previously conducted, it is estimated that an area settlement of from 18″ to 24″ would result. Such settlement would be disastrous as not only would conduit connections be broken but the loss in hydraulic head would seriously reduce the carrying capacity of the existing tunnel. After various considerations, it was decided to surcharge the area and preconsolidate the underlying clay.

The surcharge material will later be used to complete the permanent fill and the various berms. This 20′ surcharge, if allowed to remain for a minimum of two years would probably produce 60 to 70% of the total settlement. Beyond this point, additional consolidation would require longer and impractical increases in time. To accelerate the time and the amount of consolidation, it is further proposed to install five deep wells, one in each corner and one in the middle of the area to be occupied by the settling tanks. Each

well will be equipped with a deep well pump to maintain atmospheric pressure conditions within the entire depth of the well.

Soil consultants report that since the rate of horizontal flow of water through clay is many times its vertical rate, it is believed that practically full consolidation can be accomplished in from one year to eighteen months. To check the rate of consolidation and the actual time the surcharge must be allowed to act, extensive observations on surface and sub-surface settlement observation points and on piezometers will be made. In addition, laboratory consolidation tests on clay samples will be made from time to time to determine its bearing capacity.

The reason for these precautions is that it is essential to prevent settlement. For comparative purposes an unyielding H-pile foundation would cost in the neighborhood of ¾ of a million dollars; whereas, the foundation proposed can be completed for approximately ¼ of a million dollars—an apparent saving of ½ of a million dollars.

SANITARY, HYDRAULIC AND MECHANICAL FEATURES

By Elson T. Killam*

Basic Alterative Considerations in the Design of the Treatment Works

The policy of providing primary treatment and chlorination together with digestion of sludge and the pumping thereof to President Roads in coordination with the South Metropolitan District Commission Works, was established several years ago as a result of preliminary engineering reports.

The required capacity of the proposed works was established as follows:

Design Population	500,000
Sewage Flow-Average Annual	85 mgd
Average Dry Weather Flow	74 mgd
Ordinary minimum flow	40 mgd
Ordinary maximum flow	110 mgd
Maximum Storm Flow	212 mgd

Several basic alternatives were considered in the development of the fundamental design of the works, including the following:

1. Location of screening and grit removal facilities before vs. after pumping.

   These facilities will be provided ahead of the pumps because of the importance and magnitude of pumping operations and consideration of:

   (a) A record of extensive wear of pumps.
   (b) The large amount of grit and debris discharged from the tributary combined system.
   (c) Recognition that provision of adequate pumps in reducing deposits in main interceptors, will greatly increase the amount of grit delivered to the station.

2. Location of screening and grit removal facilities in basement of existing pumping station vs. location in a new independent structure.

*Consulting Engineer, New York City.

The basement of the Calf Pasture Station appeared to offer possible economies in construction of grit and screening facilities.

Detailed studies indicated that limitations due to elevation, uplift, together with hydraulic and structural considerations, and high cost of remodeling and maintaining flow were such as to demonstrate that a new independent structure would be most advantageous.

3. Remodeling existing station vs. construction of new pumping station.

A new station will be provided because of the unadaptability of the existing structure, designed for massive steam pumps, and due particularly to the high cost of remodeling and difficulties of maintaining flow during construction.

4. Alternate Locations for Settling Tanks and Sludge Works.

The most important alternate study involved location of the settling tanks and sludge works on Moon Island vs. the location of these units at Calf Pasture.

The Moon Island Location was obviously advantageous in that it was more remote and involved less difficult foundation problems.

The Calf Pasture Location, however, offered many advantages including:

(a) Discharge of treated instead of raw sewage into Dorchester Bay in event of interruption of operation of the 60-year-old tunnel.

(b) Improved carrying capacity of the Dorchester Bay Tunnel anticipated if settled and chlorinated vs. raw sewage is carried, inasmuch as the tunnel comprises the controlling factor in the capacity of the discharge works, the capacity thereof is of primary importance.

(c) Effective utilization of sludge gas for pumping.

(d) Consolidation of supervision, mechanics and operating forces, involved in treatment and pumping.

(e) Availability of R.R. siding for tankcar delivery of chlorine vs. trucking to the remote Moon Island site.

(f) Convenient access for personnel.

(g) An estimated saving in total annual expense of $150,000 per year, over other alternates.

In the development of the design the works as now detailed reflect careful consideration and adoption of the following salient factors:

(1) Diversified control of influent lines because of the vulnerability of the low level screen-grit removal operating floor and to provide means for flexibility and accurate proportioning of flow among the multiple units.

(2) The adoption of mechanical rakes for cleaning coarse bar racks in view of the large diameter and extensive system of combined sewers tributary to the plant.

(3) The selection of multiple (8) grit chambers and incorporation of Camp regulators to expedite velocity control within optimum limits for grit deposition.

(4) The adoption of long rectangular grit channels for structural reasons due to channels being over 30′ below ground level.

(5) The provisions of extensive pumping facilities because of the wide variations in flow 40 to over 200 MGD, and because of the importance and advantages of keeping the Boston Main Interceptor free from backflooding.

(6) The storage and utilization of sludge gas for peak flow as well as normal pumping, thus providing means for tripling the rate of pumping with no appreciable added cost of operation.

(7) Provision of automatic float control in coordination with engine throttling to reduce manual operation of engines.

(8) The use of manifold type pump discharge flumes to provide fixed discharge level under all rates of pumpage.

(9) The use of hydraulically balanced influent flumes to expedite equal distribution among the 3 settling tanks.

(10) The use of traveling scraper type sludge removal mechanisms in order to minimize the extent of moving parts submerged in sewage and to reduce need for dewatering the relatively large tanks.

(11) The provision of effluent control works to maintain submergence of chlorine diffusers and to minimize odors resulting from agitation of the effluent passing over effluent weirs. This problem arises from the long outfall and wide fluctuations in level under varying rates of discharge through the tunnel.

Summary Description of Proposed Works

The proposed works will include automatic influent control, mechanically cleaned trash racks, mechanically raked fine bar racks with screenings grinders; rectangular grit chambers with scrapers, inclined screw type grit washers, belt conveyors and pneumatic ejectors for discharge of grit, together with Camp type regulators for velocity control. all located in deep structures ahead of the pumping station.

Sewage pumping facilities will include three gas engine driven and one motor driven 70 MGD pumping units with one 10 MGD and one 30 MGD motor driven float controlled units to deliver sewage from the wet well to the settling tanks.

Pumps will discharge to large aeration channels equipped with swing type diffusers, with the flow then distributed to three settling tanks, with two traveling scrapers arranged to be operated in any of the three tanks.

Settled effluent will be chlorinated and measured by venturi before discharge to existing outfall works.

Sludge will be pumped from settling tanks sumps to two stage digestion tanks and digested sludge will be pumped through a force main across the bay, across Thompson's Island thence across the outer bay to Long Island where it will join the sludge line from the Nut Island plant leading to President Roads.

Screenings will be ground and immediately returned to the channel; grit will be pneumatically discharged to fill through a movable surface line on adjoining low areas. Grease will be pneumatically discharged from the settling tanks to a receiver discharging by gravity to digesters or to tank trucks as desired. Sludge gas will be utilized for normal and peak flow sewage pumping, blower operation, and for heating as available.

Detailed Description of Major Units

1. *Influent Control*

In view of the fact that the invert of the terminus of the Boston Main Interceptor is 23.5 feet below mean high water, supplemental provisions will be made in connection with influent control to protect the facilities for screening and grit removal with an operating floor level at elevation 0.

Each of the two 96-inch main influent gates will be automatically float controlled by means of hydraulic cylinders actuated by compressed air from the main air system with secondary oil transfer pumps which in emergency may be electrically or manually operated.

2. *Mechanically Cleaned Coarse Bar Racks*

Difficulties have been experienced in cleaning debris from coarse bar racks in large plants, even in the case of separate systems, during periods of extreme peak flows. Because of the extensive system of large combined sewers tributary to Calf Pasture, it was deemed advantageous to provide mechanisms for this service.

After examination of equipment in storm water pumping systems and other types of service, a traveling rake of the type developed for cleaning trash racks at power plant intakes was selected for the design. The unit, running on rails normal to the trash rack channel, may be manually propelled to position in front of either trash rack. A motor-operated hoist will actuate the rake which will deposit the trash in the traveling hopper body of the mechanism, from which it will be discharged on to a sorting table with the heavy trash separately disposed of in buckets and with the lighter trash raked to adjacent grinders for return to the adjacent influent channel.

3. *Mechanically Cleaned Fine Bar Racks*

Four mechanically cleaned fine bar racks, each 7′6″ gross width, will be provided, with individual grinders for each rack. Clear spacing will be one inch and the raking mechanism will be arranged for periodic interval operation, supplemented by automatic float control which will predominate in the event of excessive head loss through the rack. The discharge from the screenings grinders will be immediately returned to the underlying screen channel.

4. *Grit Chambers*

Six rectangular grit chambers will be provided, the shape adopted being influenced by structural considerations due to the fact that the inverts of the channels will be 33 ft. below grade. Each channel will be 85 ft. in length, with a depth at high water of approximately 8 ft., and a width of 10 ft. Grit will be collected with conventional flight conveyors supplemented by inclined screw conveyors and washers discharging to two horizontal belt conveyors which, in turn,

discharge to a battery of four pneumatic ejectors. The grit will be discharged for fill through a portable outlet pipe on the site terminating in a nearby marsh. The grit chambers will be provided with "Camp" type controllers in order to maintain grit chamber velocities within a suitable range. The entire operating floor of the grit screening chamber room will be at elevation 0, with the water level 7.5 ft. below high water and 11 ft. below low water.

5. *Pump Well*

The pump well will be approximately 3,000 sq. ft. in plan with a normal range in water level of 2 ft. Because of the substantial area of wet well required in conjunction with the high pumping rates during maximum flow, and in order to eliminate maintenance difficulties which have occurred by deposition of grit in the old station, supplemental facilities will be provided for the convenient removal of solids in the pump well. The suction from one pump will be carried to a depth of 9 ft. below the floor of the wet well. By manipulation of the gates the wet well can be dewatered and conveniently flushed by high rate discharge from the aeration channels immediately overhead and approximately 50 ft. higher in elevation.

6. *Sewage Pumping Facilities*

The pump facilities will comprise one of the most important features of the entire work, as only by adequate and flexible pumping equipment can the level of the Boston Main Interceptor be controlled. This question is of prime importance, not only to minimize deposits in the main interceptors, but also to provide space for intercepting the first flush at the beginning of storm periods and thereby avoid immediate overflow through the regulators into the Harbor.

Due to wide range in pumping requirements; namely, from less than 40 to well over 200 m.g.d., considerable flexibility will be required in regard to available pumping rates. Proposed facilities for sewage pumping include three 70 m.g.d. vertical sewage pumps driven through right angle gears by dual fuel supercharged engines. The pumps will be approximately 42 inches in size operating at 360 r.p.m. and discharging 49,000 g.p.m. against a total dynamic head of 52 ft.

The three dual fuel engines, rated at 750 h.p. each, will drive the main pumps. These engines operate on either Diesel oil or sludge

gas, or on any mixture of both fuels within the range from 100% Diesel oil to 90% sludge gas, and 10% Diesel oil. The fuel proportions may be varied by the operator by means of a simple control. The engines will be provided with turbo superchargers operated by exhaust gases.

After a thorough investigation of the relative merits of the supercharged versus atmospheric engines, it was decided to take advantage of the saving in weight and size, and of the increased efficiency possible by use of supercharged engines.

Engine auxiliary units will be arranged to provide the maximum practicable sludge gas utilization.

Exhaust gases from the engines, after passing through the supercharger, will be used to supply heat for digester heat exchangers and building heating system. Gas fired boilers arranged to burn either sludge or gas or city gas will be provided to supply additional heating capacity as required.

The dual fuel engines will be automatically controlled from the wet well through governors and additional flexibility will be provided by the provision of the following facilities:

a) A manually controlled motor drive 70 m.g.d. standby pumping unit.
b) A 10 m.g.d. motor driven centrifugal pump.
c) A 30 m.g.d. motor driven centrifugal pump.

The last two units will be automatically float controlled and will assist in maintaining a relatively uniform well level and will reduce the necessity for undesirable frequent variation in operation of the main engines.

In order to obtain more uniform pumping conditions with a wide range of flow varying from an estimated minimum of 40 to a maximum of over 200 m.g.d., the pump discharges will be provided with tapering discharge manifolds terminating in wide shallow outlets all with inverts at elevation 32.64. In other words, regardless of the aggregate pump output, either unit will at all times discharge against a definite elevation which, coupled with a normally minor variation in wet well level, will allow the careful selection and efficient performance of pumping equipment.

Control System

For maximum economy of operation, it will be essential to coordinate the pumping operations as closely as possible with the water levels in the grit channels and to avoid increased pumping heads due to lowering the wet well level more than necessary to provide free outlet for the grit channel control sections.

In order to facilitate operation and control of the plant and in order to provide full operating records, careful attention has been given to the location and operation of control equipment and gages.

A main control center in the pumping station will provide coordination of all essential plant operations. The control center will include the main gage panel on which all information required for operation of the main plant units will be readily available in the form of large dial indicating and recording gages, an electrical panelboard mounting indicating lights and push button controls for the main pump valves and other remotely controlled electrical equipment, and the control board for the two-way plant intercommunication system.

These centralized controls will enable the operator to check at a glance the operation of the main units of the plant. Water level gages will be provided to measure all important points in the plant hydraulic profile. Alarm signals will be provided for important portions to eliminate the possibility of error. The operator may, in addition, control the operation of any unit of plant equipment by means of either the conveniently located push button controls for remote controlled units, or by use of the intercommunication system. Main dual fuel engines will be controlled from operating panels, located near each engine on the same floor as the control center.

Aeration Channels

Due to the substantial waterway required for the pump discharges, the problem of avoiding settling and formation of sludge pockets under widely varying conditions of flow, and furthermore in order to provide a short term period of aeration prior to settling, the pumps will discharge into aerated channels providing a detention of approximately 6 minutes at an average daily flow of 74 m.g.d. The channels will be provided with swing type diffusers with an air supply of 3,400 c.f.m. furnished through a gas driven blower with alternate facilities consisting of a motor driven blower of equal size.

Settling Tanks

Sewage will enter the settling tanks through influent channels with waterway designed for balanced hydraulic losses to expedite uniform distribution, among the three units. The three settling tanks will be approximately 210 ft. long, each 75 ft. wide providing a theoretical detention of 1½ hours at average flow of 74 m.g.d.

High water in the settling tanks will be at grade 32.2 or approximately 46 ft. above the invert of the Boston Main Interceptor.

Mechanisms submerged within the settling tanks will be limited to cross collector in the channel across the influent end of the tanks and a traveling surface skimming mechanism at the effluent end of the tanks.

The removal of sludge from the settling tank will be accomplished by traveling crane type mechanisms with a hoist actuated scraping blade for scraping the sludge to the influent end and with the blade elevated to the surface of the tank on the return for the purpose of skimming. Two machines have been provided with facilities to enable each unit to be operated in any one of the three settling tanks. The traveling speed of the unit, while scraping sludge will be approximately 2 ft. per minute, and while skimming approximately 10 ft. or less per minute.

The units will be provided with automatic stop devices at the end of travel and will have transfer trucks for shifting from tank. The equipment will be electrically operated from a third rail system with independent motors for traction and for operation of the sludge scraping blade. On the return trip, after scraping sludge, the mechanism will convey the scum to the effluent end where a traveling type skimming mechanism will concentrate the sludge at scum weirs from which point it will flow to pneumatic ejectors for transfer to a receiving tank and ultimate disposal to digesters or to tank truck.

Chlorination

Provision will be made at the outlet of the settling tank structure for the application of chlorine through submerged diffusers. Chlorination facilities will involve tank car delivery on an adjacent siding, five evaporators and five chlorinators each with a rated capacity of 6,000 pounds of chlorine per day and other appurtenances including automatic program control and twin booster pumps for supplying high pressure water for the chlorinator injectors.

Venturi Meter and Outfall Control Works

Sewage leaving the settling tanks will, after chlorination, pass through a venturi meter 10 ft. by 5 ft. and will terminate in a control chamber which will normally discharge sewage through one of the existing deposit sewers, thence through the Dorchester Bay tunnel, to Moon Island.

An automatic hydro-pneumatically operated bypass sluice gate will discharge excess flows after chlorination to the second of the existing deposit sewers with the discharge of such excess storm flow at the terminus of the deposit sewers or, in other words, at the west shaft of the tunnel.

The outlet control works will differ considerably from a normal installation due primarily to the varying coefficient in the Dorchester Bay tunnel and the accompanying wide range of flow which may be handled. In order to pass as great a proportion of the flow as possible, through the Dorchester Bay tunnel, even during storm periods, it will be necessary to take full advantage of the maximum proper operating level at the west shaft. During low flows, the gradient at the west shaft would lie below deposit sewers into which the sewage from the plant will be discharged and it was considered essential to avoid the "Miniature Niagara" which would accompany the discharge of over 50 m.g.d. over the settling tank weir with a coincident drop of 15 ft.

Sludge Disposal

A policy of sludge disposal involving digestion and pumping to the South Metropolitan Sludge Outfall on Long Island was established as a result of earlier engineering investigations. Obviously with this method of sludge disposal the capacity of digestion tanks would be less than that required if the digested sludge were to be dewatered, however, the appreciable economic advantages of gas utilization resulted in the adoption of substantial digestion capacity aggregating 450,000 cu. ft. or the equivalent of approximately 0.9 cu. ft. per capita.

In order to provide maximum reduction of bacteria incidental to digestion, in order to provide maximum gas production with a low per capita digester space, and in order to provide maximum flexibility for the future in which event that some form of sludge dewatering

might be adopted, two stage digestion was provided. Four digesters will be provided as described by Mr. Reeves. There will be provision for recirculation and four external heat exchangers each with a capacity of 1½ million BTU per hour will be provided.

It is estimated that approximately 7 million BTU per hour will be available from the gas collection system. Sludge will be pumped from the settling tanks by means of four 150 g.p.m. plunger type raw sludge pumps and discharged to either primary digesters or to the heat exchangers.

Digested sludge will be drawn to pump and mixing chambers where optimum concentration for pumping will be developed. The sludge will then be pumped to the Boston Metropolitan sludge line at Long Island by means of one 400 g.p.m. and one 300 g.p.m. plunger type positive displacement sludge pumps. The sludge force main, 8 inches in diameter, will extend across Dorchester Bay to Thompson's Island, thence across Boston Harbor to Long Island at which point, 19 thousand ft. from Calf Pasture, it joins the 12″ Metropolitan District Sludge Outfall ultimately being discharged at President Roads.

ACKNOWLEDGMENT

We wish to take this opportunity to gratefully acknowledge the valued cooperation of the Division Engineer and staff and of the Massachusetts Department of Public Health.

STRUCTURAL AND ARCHITECTURAL USES OF ALUMINUM ALLOYS IN BRIDGES

By B. J. Fletcher*

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held April 13, 1949.)

This is the second time that I have had the pleasure of meeting with the Boston Society of Civil Engineers for a discussion of structural uses of aluminum alloys. The previous occasion was on March 10, 1937.

At that meeting we discussed a variety of aluminum structures from dredge booms to hydraulic gates. This evening, we will confine our attention to the single subject of aluminum in bridges. Possibly this change from the general to the specific indicates a decrease in flexibility as far as my thoughts are concerned. Certainly the original proposal from your secretary, that I cover the subject of "Structural and Architectural Uses of Aluminum Alloys", was flattering, but at the same time a bit overwhelming, in view of the breadth of the subject. I trust that you will pardon me for attempting to cut the title down to my size by adding the limiting phrase "In Bridges".

This limitation has two advantages, while affording a topic broad enough to illustrate most aspects of the application of aluminum alloys to structures, it permits your speaker to remain on familiar ground. At this time, when the Boston Expressway Plan is under active consideration, many of you may be interested in the most recent advances in bridge construction.

In 1937 aluminum alloys were a minor item in the kit of materials available to the bridge builder. Twelve years and a major war have improved that situation. Possibly some measure of the change can be gained from the simple fact that production of primary aluminum in the United States rose from 122,000 tons in 1936 to 622,000 tons in 1948. During that stormy period many of the uses of aluminum which had potential virtue and interesting possibilities in 1937 have become realities with substance and proven merit.

The first man to cross a creek without wetting his feet undoubtedly swung hand over hand on a vine. If his choice of a crossing

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was good, others followed thus establishing the first public convenience which might be termed a bridge. Even though he had no part in planning its structure he had at least two characteristics in common with the best of modern engineers. He chose the best available means and material to accomplish his purpose, and he had the courage to leave the muddy route which had always been followed in the past.

Since his day men have improved on that original bridge. We have lost the ability to travel any great distance by a hand over hand maneuver but what we have lost in muscle we have gained in a knowledge of materials and in methods of bending them to our will. Today we cross a creek on wood, masonry, concrete, steel or aluminum. We even cross the seas dry shod, though breathless, in a device which bears no physical resemblance to a bridge but which serves much the same purpose.

In a consideration of aluminum alloys for bridges this connection between the airplane and the bridge is less tenuous than it might seem at first glance. If it were not for this powerful incentive toward the development of materials having maximum strength per pound of weight for aircraft, aluminum might still be in the pot and pan department exclusively.

Characteristics of Structural Aluminum Alloys

Pure aluminum, dead soft and with less than one part in ten thousand of any other element, would be about as useful as molasses taffy for a bridge member. It yields under a stress of less than 3,000 p.s.i. and elongates some 60 per cent before it breaks at about 9,000 p.s.i. Of course it would stand the deteriorating effects of the weather somewhat better than taffy. As a matter of fact, few materials can compare with high purity aluminum when it comes to resisting the effects of sun and rain and sea fogs and the unpleasant mixture of dirt and gases that passes for air in our larger industrial centers.

But the plane builders needed something substantially better than a yield strength of 3,000 p.s.i. in their structural members. Through nearly fifty years of sweat, embittered by occasional tears, the metallurgists have met the requirements of aircraft designers. Today aluminum alloys, with just the right additions of zinc and copper and magnesium and chromium and with triple heat-treatment, provide the plane builder with sheet and plate and shapes which have twenty-four times the yield strength of the pure metal. That is alloy

75S-T6 which is used in our newest transport and fighting planes, an alloy with typical mechanical properties of 72,000 p.s.i. yield, 82,000 p.s.i. tensile strength and 11 per cent elongation.

Between 75S-T6 and high purity aluminum lie a variety of alloys and from these has been selected a series for commercial production each of which has some property, or combination of properties, which makes it particularly valuable for a specific class of work. It is worth noting that the selective process invariably extends over a period of years and involves the three phases of laboratory investigation, plant production and practical application.

For heavy duty structures, such as bridges, neither the strongest alloy nor the purest aluminum has been chosen. Two alloys in the upper middle range of strength offer the best combination of strength, workability, resistance to corrosion and price. These alloys are most commonly identified as 61S-T6 and 14S-T6, although some variation in nomenclature unfortunately exists between different specifications. A speaking acquaintance with these alloys will prove useful to you, since you will meet them with increasing frequency.

This evening we will not intrude upon the sacred precincts of the metallurgist. It will suffice to say that the strength, lightness and durability of these materials are attained by alloying small quantities of other elements with aluminum and by carefully controlled heat-treatment. Copper is the major alloying element for 14S while a combination of magnesium and silicon is used in 61S.

Vital statistics of these allows can be expressed by tables of physical characteristics (Figure 1). Such a table is as lifeless as a family tree but, in both cases, careful scrutiny plus some imagination will reveal potential virtues and vices. Rather than run the full gamut of properties, we will select those most important to the Civil Engineer. These include tensile strength, endurance limit, elasticity, reaction to heat and cold, resistance to atmospheric corrosion, and weight.

The tensile yield strength of 14S-T6 is closely comparable to that of structural nickel steel. For alloy 61S-T6 the yield strength is slightly higher than that of structural carbon steel. For both aluminum alloys the yield strength is between 85 and 90 per cent of the ultimate tensile strength. Both yield and ultimate tensile strengths are considered in establishing the basic tensile design stresses of 22,000 p.s.i. for 14S-T6 and 16,000 p.s.i. for 61S-T6. It is interesting

TYPICAL PHYSICAL PROPERTIES

STRUCTURAL ALUMINUM ALLOYS

ALLOY	61S-T6	14S-T6
WEIGHT, pounds per cubic foot	169	175
ULTIMATE TENSILE STRENGTH, p.s.i.	45,000	70,000
TENSILE YIELD STRENGTH, (0.2% set) p.s.i.	40,000	60,000
ELONGATION, % in 2 in. (½ in round spec.)	17	13
ULTIMATE SHEAR STRENGTH, p.s.i.	30,000	42,000
ENDURANCE LIMIT, 500 million cycles, rotating specimen	13,500	18,000
BRINELL HARDNESS, 500 kg. load, 10mm ball	95	135
COEFFICIENT OF EXPANSION, per 1°F.	0.0000120	0.0000119
MODULUS OF ELASTICITY, p.s.i.	10,000,000	10,600,000

FIG. 1.

to note that the stress strain curve of the aluminum alloys differs in form from that of structural carbon steel (Figure 2). Where such steel has a sharply defined yield point at which there is a sudden change from elastic to plastic deformation, the change in aluminum alloys is more gradual, as is true of many alloy steels and of non-ferrous metals in general. The yield strength of aluminum alloys is defined as the stress which produces a permanent set of .2%. Thus it is apparent that as in other metals, there is a small, though practically imperceptible, plastic stretching of the metal at stresses somewhat below the yield strength.

Of equal importance is the elasticity of aluminum alloys as indicated by Young's Modulus. Under load the elastic deformation of aluminum is about three times that of steel thus providing resilience which is a useful shock absorber in the case of sudden changes in load or of unequal settling of foundations. This relatively low modulus has a marked effect on the design of aluminum alloy compression members including not only columns but also beams, girder webs

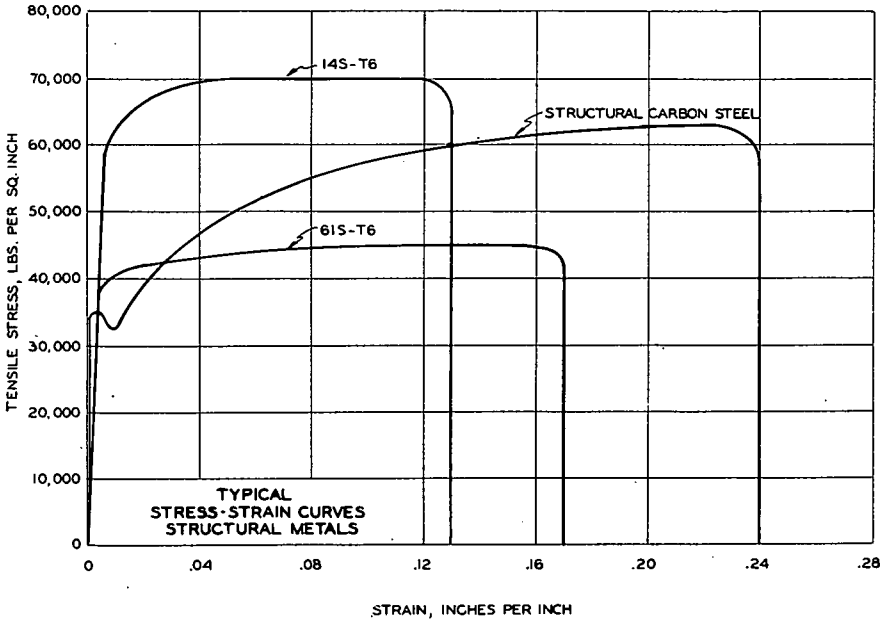


FIGURE 2.

and other elements. In general this means that economical design requires an increase in bulk of compression members as compared to steel.

The possible effect of repeated or fluctuating loads must be considered in bridge design. An economical and realistic solution of this problem requires an evaluation of the probable frequency of load application and a knowledge of the fatigue characteristics of joints (Figure 3). For standard rotating beam specimens the fatigue strength is designated as the stress which may be repeated 500-million times without failure. At this tremendous number of cycles such a stress reversal could be repeated indefinitely without causing failure. Conceivably this condition may be of interest to the machine designer but for bridges the practical number of applications of maximum loads is much less. If we consider the useful life of a bridge as 50 years and assume maximum load conditions to occur once a day we are dealing with only 18,250 cycles. If the maximum stress occurs even 100 times a day, the figure would still be under two-million cycles.

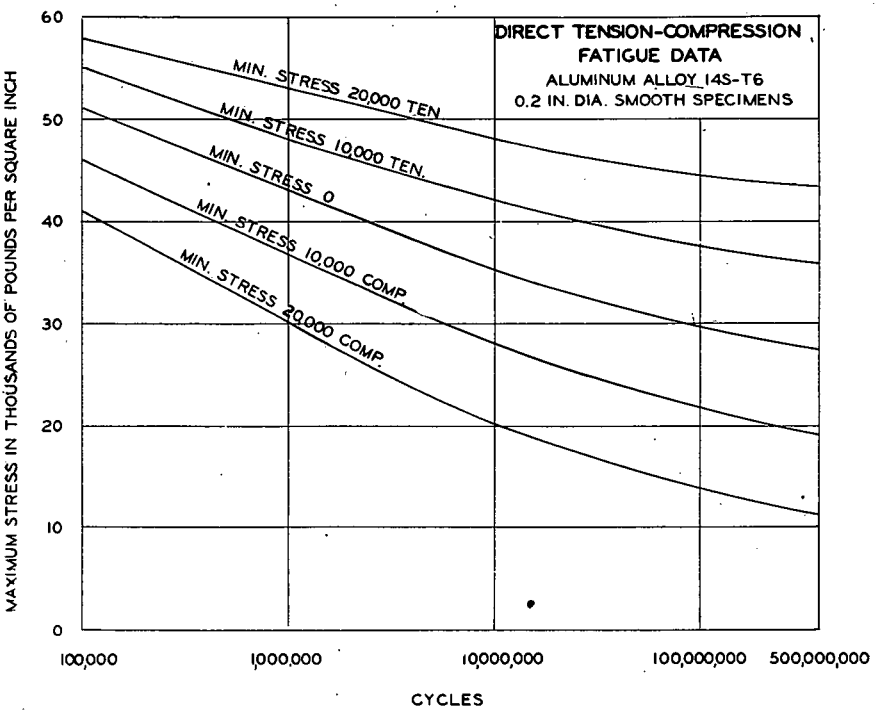


FIGURE 3

We know that discontinuities, such as sharp reentrant corners, cause a marked reduction in the fatigue life of a member. Long experience has shown that proper attention to the geometry of structural details is essential to good design in any material. Extensive tests of the fatigue strength of riveted joints in aluminum alloys have been made. Based on these tests a chart showing allowable maximum stress for various conditions of repeated loading has been prepared as a guide for the designer.

Another physical characteristic in which aluminum alloys differ from steel is in the effect of temperature on strength and elongation. A casual inspection of the curve, Figure 4, showing the variation in tensile properties with temperature indicates that aluminum alloys are not suitable for high temperature applications. Aluminum alloys are not recommended for grate bars or aircraft exhause pipes or similar items which stay very hot because at such temperatures aluminum is a puddle. Its melting point is in the range 950° to

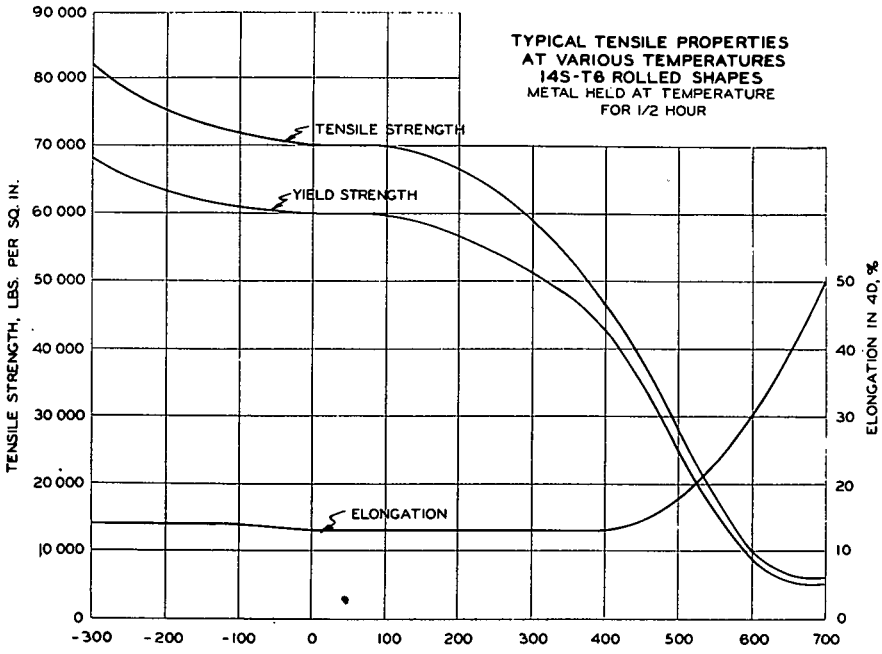


FIGURE 4

1180°F for 14S-T6. The designer is interested in the behavior of material at elevated temperatures and also in the effect of such temperatures on subsequent strength at normal temperatures.

Considering the properties at a given temperature, it is evident that aluminum likes to be cold. As it gets hotter, it becomes weaker and more ductile. After one-half hour at 500°F, 14S-T6 has lost about 60% of its tensile strength while gaining about 40% in elongation. As temperatures decrease below normal, the aluminum alloys become progressively stronger and at the same time retain their ductility. At minus 100°F both ultimate and yield strength are slightly above the values at normal temperatures and this effect continues down to the lowest temperatures at which testing has been carried out—minus 320°F. The practical value of good strength and elongation at very low temperatures is demonstrated by the uniformly successful performance of aluminum alloy aircraft frames at high altitudes and is of increasing importance in military structures for use in arctic warfare.

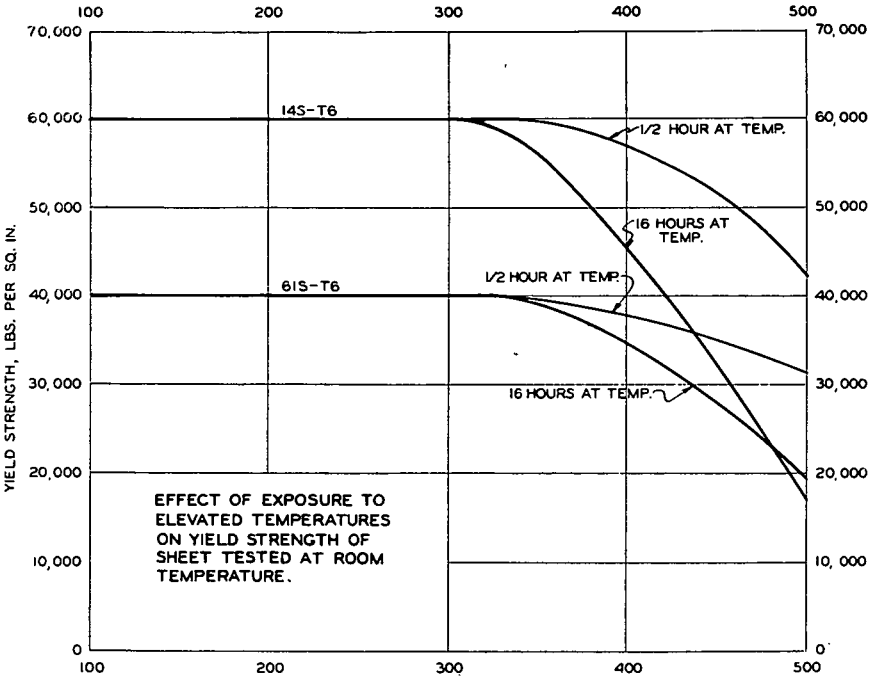


FIGURE 5

Since the structural aluminum alloys owe a considerable part of their strength to heat treatment, it is evident that strength, measured after cooling, may be reduced by exposure to elevated temperatures. Metal temperatures under 400°F for periods of 30 minutes or less have little or no permanent effect on the strength of either 61S-T6 or 14S-T6. After a half hour at 500°F samples tested at normal temperatures show a loss in tensile strength of only about 10% for 61S-T6 and 20% for 14S-T6 (Figure 5). Increases in time or temperature will cause further reductions in strength.

In all considerations of the effect of elevated temperatures it is the temperature actually existing in the metal itself which determines the effect. For short time exposure this temperature is usually substantially below that of the surrounding environment for any structure due to the high heat conductivity of aluminum.

The outstanding characteristic of aluminum alloys is light weight. Alloy 14S-T6 weighs 36% as much as structural grade steel and the

slightly lighter alloy 61S-T6, 35%. This basic comparison is easy but there is no single answer to the general question, "What is the weight ratio between steel and aluminum structures?" The answer to that depends on several considerations including size, and required performance. For a small structure such as a 50-foot span highway bridge, aluminum will save 50 to 60 per cent of the dead weight in the structural parts. In such a bridge the relation of dead load to live load is, of course, low. For a moderate length span, say 600 feet, the weight saving would be in the order of 65 to 70 per cent. In a really long span, where dead weight pyramids, the savings would approach 80 per cent. Another phase of this pyramiding of weight saving occurs in bascule or lift bridges where counterweights are required. It is obvious that the reduction of counterweight for a bascule bridge may be two to three times the weight saving in the bridge structure.

Finally the effects of the corrosive influence of atmospheres—rural, industrial or seacoast—on the durability of aluminum structures is important to the engineer. Aluminum alloys will corrode under certain conditions. The engineer must evaluate the severity of exposure at a specific location and must have sufficient background of experience, either personal or reliably reported, to decide upon the extent of protective measures required.

Alloy 61S resists atmospheric corrosion to a remarkable extent and in many localities structures of this alloy do not require painting. Alloy 14S is slightly more susceptible to corrosive influences but still ranks near the top of structural materials in this respect. Plate produced from 14S alloy may have a special surface cladding rolled on and metallurgically bonded to the base metal. This clad surface is equal to 61S in resistance to corrosion. Experimentally rolled and extruded shapes have been given this Alclad surface but present commercial production is limited to non-clad material for structural shapes of 14S alloy. In general, it is recommended that 14S-T6 bridges be given paint protection.

To summarize the properties of structural aluminum alloys 61S-T6 and 14S-T6: tensile yield strengths lie in the range of structural carbon and low alloy steels respectively; resilience is three times that of steel requiring special attention to compression members and providing a cushion against shock loads and minor foundation misalignment; conditions of repeated loadings require special considera-

tion; high temperatures reduce strength and raise ductility but both strength and ductility are maintained or rise at sub-zero temperatures; weight savings of over 50% are effected in small structures and savings of 75% or more may be possible in long span or movable bridges; substantial savings are effected in the cost of maintenance and depreciation arising from atmospheric corrosion.

These are the materials we have to work with—61S-T6 where maximum resistance to corrosion is paramount—14S-T6 where strength is the prime requisite. The next question is, "How is structural aluminum cut and joined and assembled into a useful structure?"

Fabrication of Aluminum Alloy Structures

Most of the leading structural steel shops have worked with aluminum alloys. Each has succeeded in solving the problems which arose and in each case the extent of those problems depended far more on the foresight of the designer and the ingenuity of the workmen than on any particular characteristic of the metal. An understanding of the proper method of working aluminum by both designer and fabricator, is essential to economical fabrication.

The airplane builder has specialized in aluminum construction for thirty years. Aluminum alloys fill his shop to the exclusion of nearly all other materials and his equipment has been designed around the characteristics of that metal and the extremely exacting requirements of airframe fabrication. He relies on an array of machines and equipment very different from that of the bridge builder. His everyday tools include nitrate heat-treating baths, refrigerators, stretcher presses, rubber dies, intricate machine tools, spot welders and precision assembly jigs. He even drives his rivets backwards, applying a small high speed hammer to the manufactured head and letting a back-up block and Newton's law of action and reaction form the rivet point. When he gets through he has a marvelously strong, light and accurate structure at a cost of something like ten dollars a pound, of which over 95% is fabricating cost. You can't build bridges that way and you don't need to.

The only changes a good bridge shop needs to make when starting on the job of fabricating an aluminum structure is to lock up cutting torches and arc welders, sharpen shears and punches, borrow a rivet heater with good temperature control, and possibly buy a saw or

two. Standard bulldozers, edge planers, reamers, milling machines, shears, punches and riveting equipment work well on aluminum alloys. The shapes and plate are clean and true and light to handle and the cost of most fabricating operations is just about the same as it would be for steel. Mistakes cost more because of the price of material, riveting may be a little slower where hot rivets are used but, to offset this, machining and reaming is faster and many of the light weight shapes can be moved by hand without waiting for a crane.

Admitting then that you do not have to look far for a shop which can build an aluminum bridge, let us consider briefly the few special characteristics which will influence design and fabrication.

We have already seen the effect which heat has on strength. It is obvious that severe bending and forming operations of the "blacksmithing" variety are impractical for aluminum alloys unless the part can be heat-treated after working. Moderate cold bending operations such as bevelling of angles, or rolling shapes or plate to long sweeping curves can be performed by any shop. In such work 61S-T6 will stand considerably more severe cold bending than 14S-T6. Flange angles for fish-belly girders can be bent cold but the designer should make the curves sweeping rather than abrupt. Offsetting of web-stiffener angles can be done but may require controlled heating and should be avoided if possible.

Plate and shapes ½ inch thick or less can be sheared. As with steel, edges of plates should be planed and ends of shapes milled where the best quality of work is required in principal load carrying members. Aluminum alloys cannot be flame-cut because a very ragged edge is produced with severe local damage from temperature. Special cuts are made by circular saws, band saws or milling machines.

The welding of aluminum alloys has been advanced tremendously by the recent development of inert gas shielded electric arc processes such as Heliarc and Aircomatic. Sound welds of excellent appearance are made using an atmosphere of argon or helium gas, thus eliminating the need of corrosive fluxes. The electrode is either tungsten, which is not consumed, or aluminum, which melts to form the filler material. Still, such welding is more limited in general utility than is the case for steel. Welds in either 61S-T6 or 14S-T6 give relatively low joint efficiencies because of the nature of the weld bead, which is a casting, and because of the annealing effect of the welding temperatures on adjacent metal.

For many architectural features, where strength requirements are moderate, welding can be used to advantage. Even in primary members it may be useful and it has been employed extensively on aluminum military bridges. The location and extent of welding must be controlled carefully by the designing engineer and its promiscuous use in the shop must be prohibited.

Today, and in the predictable future, riveting will be the principal method of making joints in aluminum bridges and major structures. Fifteen years ago steel rivets, hot or cold driven, were frequently used. Steel rivets had the advantages of low cost, ready availability and ease of driving but they also had major disadvantages. Paint breaks down first on steel rivet heads and unsightly rust streaks result; steel rivets are heavy and, where the engineer is fighting dead weight, the relative weight of steel rivets in an aluminum structure is discouraging; the heat from steel rivets may reduce the strength and resistance to corrosion of aluminum alloys in certain cases.

Aluminum alloy rivets are driven both cold and hot, the choice depending on alloy, required strength, and accessibility. Cold driving has been preferred in many cases because of improved shear strength and reduced costs. In 61S-T6 structures the same alloy is used for cold rivets giving a shear strength of 30,000 p.s.i. while for 14S-T6 structures a similar alloy, A17S-T4, is often used for cold rivets giving shear strengths of 33,000 p.s.i. Cold riveting is of particular virtue where long runs of rivets, such as the rivets in flange angles and cover plates of girders, can be made in the shop. Pressures required to drive such rivets are of the same order as for cold steel rivets and flat or slightly conical driven heads are recommended.

Aluminum rivets over ½ inch in diameter are difficult to drive cold with pneumatic hammers. Where hammers must be used in the field or for cramped locations in the shop, hot rivets are driven. For this type of work the rivet alloy is 61S-T4 or the related alloy 53S-T61. Only about one-fifth as much pressure is required for hot driving as for cold. Aluminum rivets are heated in a controlled temperature furnace to the heat-treating temperature, then transferred from furnace to hole and driven with the least possible loss of time. Cold metal and tools provide a quench which results in shear strengths of about 23,000 p.s.i. for 61S or 53S alloys. While this strength is only about one-half of that of steel rivets it is sufficient for many

cases and the required increase in the total number of such rivets is less than would be anticipated from a simple comparison of strengths.

Despite this it is obvious that higher strength rivets could effect savings in time and costs and such a rivet is being developed. This rivet alloy, known as XV77S, is being used for large scale experimental jobs, such as military bridges. It is driven hot, but has the advantage of a wide range of heating temperature, 850°-1000°F. Driving pressures are higher than for 61S-T4 rivets or steel but still ⅞" diameter XB77S rivets have been driven in production with pneumatic hammers using a modified cone type driven head. The shear strength increases after driving through aging at room temperature as follows: one-half hour—33,000 p.s.i., one week—38,000 p.s.i., three months—42,000 p.s.i. While this increase in strength through aging may seem strange to the structural steel man, it is a commonplace phenomenon to those who work with concrete. This new high strength rivet, representing years of careful work by the metallurgist, will be used with increasing frequency and effectiveness.

Another factor influencing design is the range of sizes available in shapes and plate. Strong alloy aluminum plates are rolled in any size which is apt to be required for girder webs, columns or cover plates. Single plates ten feet wide, ¾ inch thick, 26 feet long were produced for the Grasse River Bridge. Larger plates will be available from a new mill just going into production at Davenport, Iowa. Present shape sizes range through 8- x 8-inch angles, 15-inch channels, 14-inch wide flange beams. Deeper girder beams must be built up, though larger sizes can and will be produced when the demand justifies the investment in new equipment and mills.

Price of material is always a controlling factor in design. We shall consider overall costs later. At the moment it is sufficient to note the general effect of a price which averages about 34 cents a pound for structural material. Quite obviously, refinements of detail to eliminate needless weight are in order. Similarly a careful structural analysis is justified. Rule of thumb designing, with a selection of sizes from lists of tables compiled to cover a wide range of conditions, is wasteful. Potential savings in cost and weight offer a challenge to the best talents of the engineer, not only in the matter of detail and structural analysis but in overall evaluation of the problem. A reappraisal of conventional limitations is necessary. What is a logical restriction on deflections; what is the true probability of

extreme loads and of conditions of fatigue; why should there be a limitation on minimum thickness in a corrosion resistant material; what is the effect on form of structure of compounding reductions in dead weight; will special shapes, available by extrusion processes save cost; will tubes or hollow members be economical? The effectiveness of the answers to such questions measures the ability of the engineer dealing with structural aluminum.

A detailed listing of design formulas and procedures would be out of place here but it is reassuring to the engineer to know that a wealth of background information on the design of aluminum alloy structures is available. Reports of innumerable laboratory investigations, a complete structural handbook, design specifications prepared by bridge engineers, material producers and government agencies give guidance in design. Successful aluminum design requires study and constructive thinking, but the fundamental data are available in abundance.

Many able engineers have contributed to a better understanding of the design of heavy duty aluminum alloy structures. Specifications prepared by the late Mr. Leon S. Moisseiff marked a notable step forward; Mr. O. H. Ammann and Mr. Shortridge Hardesty have prepared detailed and effective design studies and specifications; Mr. Howard H. Mullins of the Engineer Research and Development Laboratories U. S. Army has done notable work in the design of military bridges. Fundamental research and studies conducted by Mr. R. L. Templin and Mr. E. C. Hartmann of the Aluminum Research Laboratories have provided the foundation upon which the design of heavy duty aluminum alloy structures has been built.

Examples of Aluminum Alloys in Structures

Within the last three years there has been a notable increase in activities tending toward the use of aluminum alloys in bridges. No one who has followed this development closely is laboring under the delusion that aluminum will supplant steel and concrete as the principal materials of bridge construction, yet there are powerful and fundamental forces at work which will require engineers to give increasing attention to special materials to fill special requirements. Two of these forces are increased costs of labor and general acceptance of new materials.

As the cost of fabrication and erection rises relative to the cost

of material, it is obvious that the importance of initial material prices decreases. Any change which may reduce man hours of labor in the shop, in the field, and in maintenance assumes increasing importance in overall economics. With steel at two cents a pound and labor at a dollar an hour, there are very few cases where thirty-cent aluminum is economical for bridges, but with steel at four cents and labor at two dollars, then thirty-five-cent aluminum finds additional fields of usefulness.

Public consciousness of and acceptance of aluminum was limited to a few items, such as cooking utensils, thirty years ago. During the war everyone learned of the need of aluminum for aircraft and of the excellent performance of fighting planes under the most adverse conditions of weather and damage from shell fire. Hundreds of thousands of men and women worked with aluminum in all phases from the mining of bauxite ore to the riveting of components of aircraft. This introduction has ripened into familiarity with the tremendously expanded uses of aluminum on farms, in homes and in public buildings.

On the professional level architects and engineers have expressed their confidence in the merits of aluminum through a steadily mounting volume and variety of applications and products. While a major aluminum bridge is yet to be built, and even though the number of case histories of major applications is limited, the trend toward aluminum is definite and prophetic. Let us review the story briefly.

Before discussing specific examples of aluminum in bridge structures, let us consider certain of the appurtenances and architectural features. For essential items such as railings, light standards and reflectors, toll and service houses, gates and floor gratings, aluminum alloys are available in a wide variety of products offerings good appearance and minimum maintenance.

For such applications alloys of the magnesium silicide type, of which 61S-T6 is one example, are usually chosen on the basis of maximum resistance to atmospheric attack, good strength and moderate price. Most bridges are subject to the action of atmospheres which are humid, frequently polluted with industrial smoke and fumes and not infrequently salt laden. Proper maintenance of steel parts is troublesome and expensive and since these items are frequently in prominent locations rust streaking is particularly undesirable. Aluminum alloys will withstand the attack of such atmospheres to a remarkable degree if they are properly selected and properly installed.

Severe cases of corrosion of aluminum alloys have occurred but invariably they have arisen from one or more of the following mistakes: use of alloys not adapted to service requirements; design details providing pockets or crevices where moisture will collect and be retained; contact with copper, lead, nickel or certain other heavy metals; contact with wet wood, asbestos or other absorbent materials. While the magnesium silicide type of alloy seldom requires painting on exposed surfaces it is good practice to protect surfaces contacting dissimilar materials with zinc chromate, bituminous paints or suitable mastics.

Aluminum surfaces will become dirty, like any other material, and certain atmospheres may cause mild surface pitting. A bright appearance can be maintained by periodic cleaning but even if this is neglected, the amount of damage to a proper aluminum installation will be negligible over a long period of years.

One of the most familiar uses of aluminum is as the pigment of paint. All engineers are familiar with its protective qualities and clean appearance which is as effective on aluminum structures as on steel.

Bridge railings have been used in many projects from the Atlantic to the Pacific Coasts and in many industrial cities such as Pittsburgh, Charleston, W. Va., and Cincinnati. These railings may be extremely simple, consisting of pipe with cast posts, or ornate to any degree which pleases the designer's taste. Basic costs are reasonable, in some cases as little as $10 per linear foot. Fancy decorative detail is expensive and seldom adds anything to the public appreciation of a good bridge.

Aluminum floor grating and tread plates are frequently an advantage especially where they must be manhandled or where corrosive conditions are severe. Aluminum alloy floors for pedestrian and highway traffic assure minimum weight and long life. Flat plates suitably supported and special extruded planks have been used to advantage sometimes in combination with bituminous surfacing materials. Where grating type roadway floors are desired, aluminum would have obvious advantages. A recent study of a specific bridge gave the following relative weights:

Timber deck with steel treadplates	31.6 ‡/ft^2
Steel grating, sills and stringers	24.6 ‡/ft^2
Aluminum alloy grating, sills and stringers	10.4 ‡/ft^2

As of today no aluminum grating flooring has been installed on highway bridges but it appears to be particularly well suited to projects such as bascule spans.

In recent months a number of municipalities have installed aluminum lighting standards. The objective is decreased maintenance since light weight is of minor importance, except when poles are set or replaced. Such lighting standards would be particularly suitable for bridges.

Everyone is familiar with aluminum windows for public buildings and homes. Their use on toll and service houses gives an attractive appearance and assures easy upkeep. Doors, roofing, side panels and trim in either sheet or cast aluminum offer similar advantages.

Turning to actual structural applications in bridges, brief sketches of five typical uses of aluminum will illustrate the possibilities and probable future trends.

During the fall of 1933 the badly overloaded trusses of the old Smithfield Street Bridge[1] in Pittsburgh were given a new lease on life by rebuilding the floor system using light weight aluminum alloys. The aluminum floor extends over the two main spans for a total length of 720 feet. A double track street railway, two-lane highway and two sidewalks are accommodated in an overall width of 71'-7". A total of 340 tons of aluminum alloys were used with the result that the load on the trusses was reduced by more than one ton per lineal foot. It was estimated that this reduction in dead weight would increase the useful life of the bridge by 25 years. This now appears to have been a conservative estimate as the floor system is in good condition in its sixteenth year of use.

It is reasonable to expect that certain lessons will be learned from any new venture and the Smithfield St. Bridge was no exception. The alloy used, designated as 27S-T was somewhat similar to our present alloy 14S-T6 in strength with typical tensile and yield strengths of 60,000 and 50,000 p.s.i. respectively. In most respects it was an excellent material but it had one defect in that both strength and corrosion resistance depended upon extremely accurate control of two minor elements namely an addition of tin and total exclusion of magnesium. Structurally it was tough material but metallurgically it was temperamental and consequently expensive to produce.

[1] "Heavy Bridge Floor Replaced With Aluminum," by J. P. Growdon, Ross M. Riegel and R. L. Templin. Civil Engineering, March, 1934.

Potential sources of trouble in the Smithfield Street Bridge lay in the use of hot driven steel rivets and in the fact that contacting surfaces were not painted prior to assembly. As a matter of fact no serious difficulties have arisen from these conditions. There have been minor and isolated examples of corrosion in a few joints and at occasional, apparently random, locations on the surfaces of shapes and plate. In each case the damage has been slight and correction has been a matter of routine maintenance.

Three types of structural damage have occurred. During a flood a floating derrick broke loose and its mast struck one of the track stringers causing distortion and fracture requiring replacement of the member. When the ties for the street railway tracks were originally laid the alignment, both vertical and horizontal, was poor. As a result some ties came to bearing only on the extreme toe of the horizontal leg of the top flange angles of the track stringers. This caused excessive local stressing of the angle leg as cars passed and fatigue cracks developed at the vertex of the angles in a few locations. This condition was entirely corrected by realignment and proper maintenance of the tracks. The third fault arose from the type of connection between track stringers and floor beams. Originally this connection was made very rigid and provided considerable continuity. Excessive local bending stresses were developed at the vertex of the clip angles and a few developed fatigue cracks. This was corrected by removing some of the rivets from the clips and inserting small brackets under the ends of the stringers to take end shear. When this connection was "softened up", local stresses were reduced and trouble stopped.

The Smithfield Street Bridge floor has been a good investment for the City of Pittsburgh and a valuable proving ground for aluminum alloys. It illustrates the first case in which aluminum alloys may prove economical in bridge construction, namely: the reconstruction of old bridges to make them safe for modern heavy traffic thereby extending their useful life and saving the cost of new construction.

Early in World War II it became evident that existing portable bridging equipment was inadequate for modern mechanized warfare. For a good many years the U. S. Army Engineers had been experimenting with new types of floating bridges, one of which employed aluminum boats. From this background the M-4 floating bridge was

developed. Not only boats but the entire roadway was made from aluminum. The old idea of timber stringers, or "balk", and wood planking, or "chess", was scrapped and a single member served the multiple purpose of connecting the boats, distributing wheel loads and providing the actual roadway surface. Thus the number of pieces in the bridge floor was reduced permitting much faster erection, and the hazard of a timber floor which would burn and splinter under shell fire was eliminated. These flooring members were hollow box beams made from two extruded channel shaped members of aluminum alloy 14S-T6. The top surface was provided with serrated or notched ribs to assure traction and the ends were closed so that the member would float. One outstanding departure from previous practice was assembly by welding—the first instance of welded aluminum construction in primary bridge members. Welding was confined to the neutral axis and the ends of the beams so that the aluminum alloy retained its full strength in locations of maximum stress.

The course of the war moved too swiftly to permit an extensive use of the M-4 bridge in combat but it was thoroughly tested and proven by the U. S. Engineers. The value of rugged light weight aluminum bridging equipment was demonstrated conclusively.

Lessons learned with the M-4 floating bridge have been carried on into the field of heavy duty tactical bridges during the last three years. A new bridge for the heaviest military loads on moderate spans consists of a series of pin connected trusses, massive but light floorbeams, and an ingenious all-metal deck. All principal members are aluminum alloy 14S-T6 and extensive use has been made of special extruded shapes to give maximum strength with minimum weight. Welding is use for many parts particularly in building up box sections. Main joints are riveted using the new high strength XB77S-W rivet. Fabrication of the first units of this new bridge has been completed and they have been under test for several months.

The military bridge program is of particular interest because it may well point the way for future civilian works, as military construction has done so many times in the part. The military bridge, where minimum weight with maximum strength is essential, illustrates the second case in which aluminum alloys have particular utility.

Turning back to civilian works you may recall the 100 foot alumi-

num plate girder span which was erected in the Grasse River Bridge[2] in northern New York state in 1946. The purposes of this project were: to demonstrate the suitability of aluminum alloy construction for railroad bridges; to put design and fabricating methods to the acid test of full scale production; to permit investigation of deflection, vibration and long time resistance to corrosion in comparable steel and aluminum structures subject to identical service conditions.

This project has been described in detail in the technical press so a very brief review will suffice here. The aluminum span, is of conventional design and construction except for the material which is aluminum alloy 14S-T6. It is designed for Cooper's E-60 loading and has the following dimensions: overall length 99′-7½″; width center to center of girders, 7′-0″; depth back to back of flange angles, 10′-½″. Dimensions are the same for the adjacent steel spans except that the depth is 9′-½″ reflecting the difference in flexibility of aluminum and steel.

In the fall of 1947 tests were run to determine the deflection and vibration characteristics of the bridge for both aluminum and steel spans. A class H-6-A New York Central locomotive, weighing 457,000 lbs., or 31,000 lbs. more than a standard E-60 locomotive, was used in both static tests and for tests at speeds up to 34 m.p.h. The maximum reflections under static load at the centers of the spans were 0.61″ for steel and 1.53″ for aluminum. Under moving loads these deflections increased to 1.74″ or 14% for aluminum and to .69″ or 13% for steel. The higher deflection of the aluminum span was in good agreement with calculations based on the differences in girder sections and modulus of elasticity. During the design period some concern was expressed as to the effect, either physical or psychological, of the increased deflection. No adverse effects were noted. In riding the locomotive there was no perceptible difference when crossing the aluminum span as compared to the steel spans.

Similarly considerable speculation had arisen as to the vibration characteristics of the aluminum span. The report prepared by R. L. Moore and F. E. Rebhun of the Aluminum Research Laboratories states: "The primary tensile stresses and the vertical deflections measured for both girders under moving loads of 28 to 34 miles per hour were only 12 to 15 per cent higher than the corresponding static

[2]"All-Aluminum Span Carries Rail Traffic Over Grasse River Bridge," by Shortridge Hardesty and J. M. Garrelts, Civil Engineering, Dec. 1946. "Aluminum Span for E-60 Railroad Bridge," Engineering News-Record, Nov. 28, 1946.

FIG. 6.—SETTING 100-FOOT ALUMINUM ALLOY RAILROAD BRIDGE SPAN, MASSENA, N. Y.—1946.

values. No tendencies toward resonance under the hammer blows of the unbalanced locomotive drivers were observed".

Of particular interest is the relative weights of the aluminum and the steel spans. The steel weight is 128,000 lbs., aluminum 53,000 lbs., a saving of 75,000 lbs. or over 58%. In this particular project the principal effect of weight reduction was the elimination of virtually all field fabrication with a substantial saving in time and expense. The complete span was fabricated in the Bethlehem Steel Co. shops near Pittsburgh, Pa., and shipped to Massena, New York, a distance of about 575 miles, in one piece. At the site the aluminum span was unloaded and set in place in about a half a day (Figure 6). Due to their weight the steel girders had to be shipped and set individually and the cross bracing assembled and riveted in the field. This operation took about two days per span.

As of 1946 the aluminum span cost about 70% more than steel, erected, so it is obvious that aluminum has no economic justification in such short fixed spans and indeed it is doubtful whether such

aluminum spans will ever prove economical. However, as span length increases, the cost comparison becomes more favorable because dead weight breeds dead weight. If we assume a cost for aluminum bridge structures at 65¢/lb. as against 18¢/lb. for structural steel, equal first cost will be realized when the weight of the aluminum structure is about 27% of the steel. In general this point would be reached for fixed spans about 700 ft. long. The picture is still more favorable to aluminum construction for component parts of long spans such as a suspended span in a cantilever bridge or the floor and stiffening trusses of a suspension bridge.

The Canadian town of Arvida, Quebec,[3] is building a highway bridge of aluminum alloys over the Saguenay River. The central arch span is 290 feet long and the total length including approaches is 504 feet. In this case the weight saving is stated as "over 50%". Recently Mr. O. H. Ammann prepared a detailed design and study for an aluminum arch bridge with a central span of 600 feet. In this case the weight saving was estimated to be approximately 65 per cent.

These examples indicate the third case in which aluminum alloy construction may prove economical, namely, in long span bridges where weight saving is cumulative.

Finally we come to the case of movable bridges of the bascule and vertical lift types. In this field our British friends have pointed the way by building an aluminum bascule bridge at Sunderland, England[4] (Figure 7). This double leaf trunnion bascule, completed in November, 1948, by Head Wrightson & Co., Ltd., has a length of 121′-1½″ between trunnion bearings. It accommodates a nine-foot roadway and standard gauge track plus two sidewalks. In any bascule it is obvious that weight saving in the section ahead of the trunnion will permit large reductions in counterweight requirements with consequent savings in operating machinery and possibly in foundations. The ratio of savings in counterweight to savings in structure may be as high as 3 to 1. Savings in counterweight metal and space are particularly valuable for fairly long leaf bascules.

A similar situation prevails in lift bridges but here weight reduction in counterweight is essentially equal to the suspended span. Additional savings are effected in tower steel, cables, machinery and possibly in foundations. In one study of a 300-foot lift span with a

[3]The Engineering Journal, April, 1949. "The Arvida Bridge," by C. J. Pimenoff.
[4]"Hendon Dock Aluminum Bridge." The Engineer, Dec. 3, 1948.

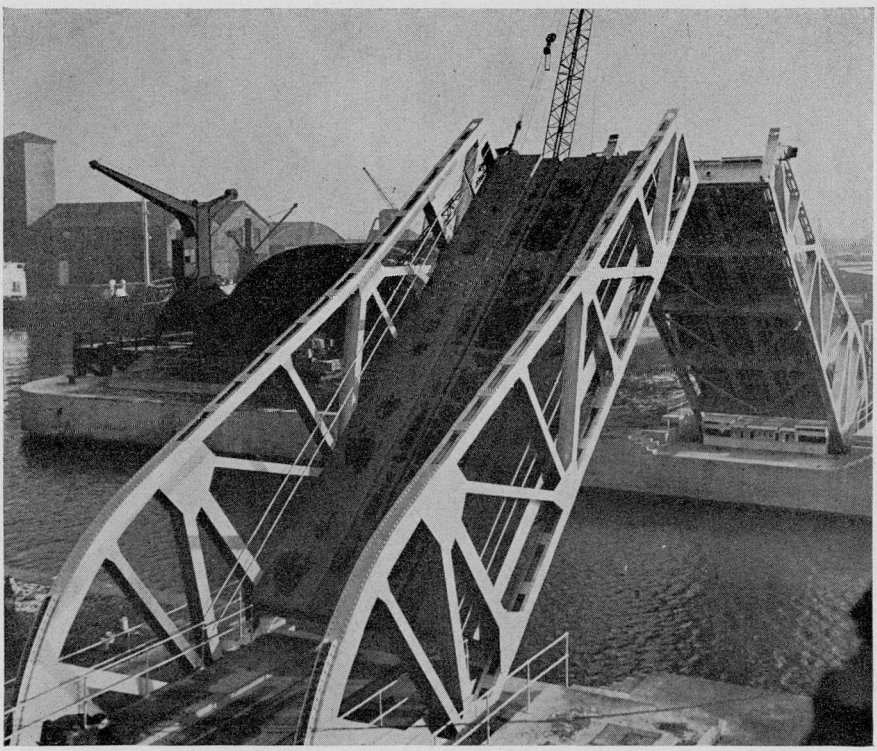


FIG. 7.—ALUMINUM ALLOY BASCULE BRIDGE, SUNDERLAND, ENGLAND. ERECTED 1948 (PICTURE BY COURTESY OF HEAD, WRIGHTSON AND COMPANY LIMITED).

low ratio of live to dead load the saving in weight of the lift span was 70% with an equal saving in counterweight plus a 50% saving in cable and machinery weights.

The movable bridge represents the fourth case in which aluminum alloys may contribute to the economy of bridge building.

For the past ten years extraordinary military and civilian demands have required the utmost efforts of the several producers of aluminum alloys to keep up with demands for this light metal. During that time the availability of structural aluminum products has been severely restricted. In spite of that, great progress has been made toward a better understanding of the material, and toward improved products, alloys, and fabricating methods. The possibilities of weight saving and reduced maintenance costs have been demonstrated by numerous projects. In future years, with conditions of

supply returning to normal, bridge engineers will make use of structural aluminum alloys to an ever-increasing extent.

Professor Dietz suggests that further discussion of the resistance to corrosion of structural aluminum alloys would be interesting.

Much time and effort has been devoted to such corrosion studies and results have been reported in numerous papers, two of the best of which are "The Resistance of Aluminum-Base Alloys to Atmospheric Exposure",[5] and "Resistance of Aluminum-Base Alloys to Marine Exposure"[6]. In one series of tests .064 inch thick A.S.T.M. sheet tensile specimens were exposed at several locations and the change in tensile strength was measured to determine the relative resistance to corrosion. At Point Judith, R. I., in an exposure station surrounded on three sides by the sea, alloy 61S-T6 lost 9% in tensile strength in 8 years, alloy 17S-T4 (which is closely comparable to alloy 14S-T6 in resistance to corrosion) lost 12% in 10 years, steel lost 100% in 2 years. At New Kensington, Pa., which has a typical industrial atmosphere, both 61S-T6 and 17S-T4 lost 8% in 10 years, steel 85% in 10 years. Since corrosive attack starts on the surface, the percentage decrease in strength of bridge material of normal thickness will be much less than that of these 1/16 inch thick specimens which have a large surface area compared to their thickness. Also, the rate of attack generally decreases with time for aluminum alloys.

It has been said, most aptly, that "corrosion of metals is a problem calculated to confound the experts." Categorical statements concerning behavior are justly suspect because of the great variety of conditions which may be encountered in service. It is always necessary to supplement general rules with a thorough appraisal of local conditions to determine specific requirements for any sizable project. With this limitation in mind, the results of past experience lead to the following conclusions and recommendations in answer to the questions asked by Prof. Dietz.

1. Structures of alloy 14S-T6 should be painted. The exposed surfaces of structures of alloy 61S-T6 do not require painting for any normal atmospheric exposure including most industrial districts and seacoast exposures beyond the reach of frequent spray.

[5]E. H. Dix, Jr. and R. B. Mears, published by the American Society for Testing Materials, 1946.

[6]R. B. Mears and R. H. Brown, 1944 Transaction, The Society of Naval Architects and Marine Engineers.

2. The use of hot driven steel rivets in aluminum alloy structures is not recommended. Aluminum rivets should be used. Galvanized and stainless steel bolts have given satisfactory results under normal conditions of atmospheric exposure.

3. Moisture should be excluded from all joints in aluminum alloy structures and from contacting surfaces between aluminum and dissimilar metals, concrete or wood. This can be done by using mastic materials or synthetic rubber gaskets. Many satisfactory caulking compounds and gasketting materials are available and specific recommendations can be obtained from producers of aluminum structural commodities. Compounds containing lead or copper asbestos or absorbent materials should be avoided.

4. Aluminum alloy structures either constantly or intermittently submerged in fresh or salt water should be painted.

5. Proper painting procedure includes: cleaning to remove dirt, oil, grease and other foreign substances; priming with zinc chromate paints; applying two or more finish coats of aluminum paint or other high grade bridge paints.

RADIOACTIVE TRACERS IN FLOW TESTS

By Ralph S. Archibald*

REPORT OF STUDY UNDER SCHOLARSHIP FROM JOHN R. FREEMAN FUND

(Presented at a meeting of the Hydraulics Section of the Boston Society of Civil Engineers, held on November 2, 1949.)

Synopsis

In 1923 C. M. Allen and E. A. Taylor developed the Salt-Velocity test to its first practical use in measurement of the flow in water power penstocks. Since then many adaptations of the method have been investigated and tested. The same may be said of the Salt-Dilution method as developed by Groat. However, both methods failed in sewage due to chemical action on the salts or dyes used. Density problems were encountered in some instances especially in basin testing. The accurate use of these tests was then generally limited to the testing of large, high velocity conduits by the Allen method.

However, the use of a radioactive tracer eliminates many of the stumbling blocks that have held up use of the tests with dye or salt. The new tracer is unaffected by chemical or physical changes in the solution and density effects are improbable due to the minute quantities required of the radiotracer. Further, in model testing and in tests with small pipes, the Geiger tube that picks up the radiation needs only to be placed on the outside of the pipe and the tracer followed and its distribution curve obtained as it passes under the tube. In this way no samples need to be taken and the flow pattern remains undisturbed. With large conduits this method is, as yet, impossible as very large amounts of radioactive material are required. For conduit measurements about one millicurie per minute per c.f.s. is required for tracing through the pipe walls. For larger pipes then, sampling must be resorted to and in very large penstock-type conduits this is a difficult and probably inaccurate procedure. Therefore in pipes of this size the Allen method is the only accurate test available. Smaller pipes and sewers, though, can be accurately gauged with ease using the radioactive tracer.

In basin tests to determine volumetric efficiency and other basin characteristics, the radioactive tracer is very useful where dyes and salts formerly gave poor results. Here, a more complete flow-through curve is obtained and the various parameters determined with greater accuracy. Field tests are possible and reasonable amounts of radioactive material are all that is needed for testing even large basins such as ponds. Models offer a method of design and using this new tracer the testing of models is extremely easy, accurate, and satisfactory. The effect of various baffles can be studied and a proper design selected in this way. Short circuiting is an important problem and this can be efficiently studied for the design of such tanks as are used for chlorine contact purposes, for example.

The radioactive tracer, it is believed, will find its place among the tools of the hydraulic and sanitary engineer and will uncover many new factors of hydraulics and sewage and water treatment that have hitherto been hidden.

Introduction and Objectives

For many hundreds of years the measurement of flowing water has been a problem for hydraulic engineers. The need for this type of measurement has been very great, for such reasons as metering for water supply and irrigation. For relatively small flows in pipes and other closed conduits of ordinary size this problem has been fairly well solved by the use of such meters as the Venturi which give good precision with a moderate loss of head. However, for such conduits as those of a water power station where the flows and diameters are so large and the head required for a meter too valuable, few satisfactory methods of flow measurement exist. The flow of sewage offers another problem as yet poorly solved, because the large floating solids tend to clog a permanent meter. The measurement of rivers, canals, etc., can be done only with a great deal of effort and expense. The flowing-through period of tanks is of great interest to sewage and water treatment plant designers, but it cannot, as yet, be accurately evaluated. The detention periods of ponds has yet to be measured, because of the huge amount of chemicals required. These are the types of problems in liquid measurement that a new tool to be discussed in this paper will help to solve. A complete solution for all the problems has not been attained, but this paper may point the way toward future solution by others. The need for such a solution is evident.

The new technique involves the use of a radioactive element as a tracer in flow tests. Previous to the discovery of this new tool the hydraulic engineer when faced with the problem of metering a flow where a fixed meter could not be installed was forced to use either a current-meter or a dye or a salt as a tracer. The current-meter is not particularly accurate, being subject to local velocity effects. The dye is not too successful, as in sewage, for example, where it may be acted upon by the other chemicals present. Salt has a great drawback as a tracer in that large quantities are required; in some cases density currents are induced that distort the flow pattern. The salt technique is also not very practical in sewage because of the widely varying concentrations of chlorides already present.

Since the radioactive tracers will perform the same function as the dye or salt it will be the main objective of this paper to show where the radioactive tracers are practical and how much more efficient and usable they are than the dye or salt.

The main advantages of this new type of tracer are: (1) Radioactive compounds emit distinctive rays that may be positively identified regardless of the chemical and physical composition of their solvents; (2) Identification of such radiation is easily, quickly, and accurately accomplished; and (3) The concentrations of radioactive material required for identification are much smaller than those required by chemical methods (dye or salt).

These advantages will allow then the measurement of the rate of flow from a river to a trickle by the addition of a few cubic centimeters of radioactive material. This minimizes any possible effects from density currents; the radioactivity will perform as well in sewage as in water; and in some cases, for example in small models the measurement can be made from the outside of the pipe walls with no disturbance of flow whatsoever. These and other advantages will be stressed throughout this paper.

The History of Flow Measurement by Chemical Means

The method of flow measurement to be considered in this paper is a refinement of the present methods of measuring flow by the use of chemicals, usually salts or dyes. For that reason a short history of the method to date, together with a description of the techniques employed, will be considered in this chapter to enable the reader to see better the advantages of the method set forth in this paper, namely the use of a radioactive element as a tracer.

Development of the use of chemicals (salt etc.) as a means of measuring the flow of liquids is exclusively a product of the twentieth century. Many chemicals have been used and methods developed. These can, however, be generally broken down into two distinct types of tests. These two techniques are known as the Salt-Velocity method and the Salt-Dilution method. The word "salt" is here employed as a general descriptive term for all chemicals including dyes; sodium chloride is most commonly used as it is cheap and available. The Salt-Velocity method will be discussed first.

The Salt-Velocity Method in Conduits

The Salt-Velocity method as used in pipes and other conduits, involves the addition of the salt as a slug injected quickly into the flow at an upstream point. Downstream at two stations some distance apart the salt concentrations are recorded continuously (or at short time intervals) as the salt passes by. The time interval between the centers of gravity of the two concentrations versus time curves is then computed. Thus, the time required for the salt to pass between two stations and the distance between the stations having been determined the velocity can be computed. The area of the conduit being known the flow is easily obtained.

Until 1923, however, although this method had undoubtedly been considered in a crude way, no tests to ascertain its practicality had ever been performed, as far as is known. In that year Charles M. Allen and Edwin A. Taylor (1)* first published the results of their experiments with this method made both in the laboratory and in the field. These tests proved the worth of the method in such a complete and satisfactory manner that there has yet to be a criticism or correction made in their technique. Their methods were based on the fact that the conductivity of water is increased by the addition of a salt. Thus, if electrodes are inserted in the flow at two points, the readings of conductivity can be made by a galvanometer, and as this reading of conductivity is proportional to the concentrations of salt passing, the times can easily be computed and the velocity obtained as described before. The tests were performed in penstocks and similar pipes with high velocities and Reynolds Numbers. Many tests were performed both in the laboratory and in the field that showed the error in these velocity measurements was extremely small

*Numbers refer to references at end of text.

and many times better than any other known method for high flows without the use of permanent type meters.

This method, often called the Allen Method, is now widely accepted as a means for measuring the extremely large flows at high velocities encountered in water power, water supply, and other work.

For example in a test of the Delaware water tunnel over a distance of 23 1/3 miles in a pipe 15 feet in diameter errors were only 0.05% to 2.1%. No difference was noted between salt added in solution and adding it in the dry state. Five hundred pounds was required for each test.

The longest distance ever reported gauged was 44.5 miles in a 13.5-foot tunnel. The results here were also very satisfactory. Many other tests of this kind have been carried out all over the country and the world, but either they have not been reported or they have shown nothing new except to prove further the utility of this method.

One element common to all the above tests was high velocity or Reynolds number. The effects of low velocity were investigated by L. J. Hooper (2). In three series of tests on a 40' pipe, 12" vertical pipe, and a horizontal 2" pipe, it was found that there exists a critical velocity below which good mixing does not take place and large errors are introduced into the velocity measurements, also that the effect of gravity is negligible only so long as proper mixing occurs. He found that as the mixing energy is proportional to the velocity cubed, there is a rapid change from good to poor mixing over a relatively short range of velocities. Many others including Barnes, Fejer, Daily, and Mason have done much work in this field but much more remains to be done. Martin A. Mason (3) conducted some investigations into the theory of the Salt-Velocity tests in conduits. His investigations into the theory of the Salt-Velocity tests were limited solely to the turbulent range. He investigated the necessary mixing length required to reduce an irregular salt cloud to a Gauss distribution, and many other important facets of the test.

Thus, the history of the Salt-Velocity test in conduits can be reduced simply to the fact that the technique developed by Allen is the basis for all tests up to the present time. It is extremely accurate where the velocities are sufficient to give good mixing, but below a critical velocity its value disappears.

The Salt-Dilution Method in Conduits

The Salt-Dilution method as applied to conduits consists of adding a constant amount of a solution of salt of known concentration to the main flow that is to be measured. Samples of the mixed flow are taken downstream after a steady state has been reached and the concentration determined. Then,

$$\frac{Q(\text{main flow} + q_c(\text{added salt flow})}{q_c} = \frac{C_1(\text{conc. of salt flow})}{C_2(\text{conc. mixed flow})}$$

or $Q = q_c \left(\frac{C_1 - C_2}{C_2}\right)$ or as C_2 is very small relative to C_1

$$Q = q_c \left(\frac{C_1}{C_2}\right)$$

The most comprehensive original paper on the subject was published by B. J. Groat in 1916 (4) making this method slightly older than the Salt-Velocity method. This paper covered, with extraordinary detail, all the factors involved in this work. The total amount of salt required by this method is very much greater than in the Salt-Velocity method, but the density effects are not as great once the steady state has been reached.

Barbagelata (5) in 1928 reported on tests made in Italy using this Salt-Dilution technique. He found that a dye is useful in the test to aid in determining time ranges of flow. He also used electrical conductivity measurements of the salt solutions, finding them much easier and more accurate than chemical titrations. Some of the difficulties he found in using this test in streams were: (1) the presence of air bubbles, (2) the irregular distribution of the solution in the total volume, and (3) the irregular distribution of the solution as to time. He found that even after passage through a power plant there was still some error of measurement due to inadequate mixing. These and other suggestions helped greatly to improve the techniques of the test.

This Salt-Dilution test is not widely used because of the many difficulties, such as the large quantities of salt involved and the elaborate dosing and sampling. When it is well performed however, the results are as good as, if not better than, the Salt-Velocity method, especially for stream flow measurement.

The Salt-Velocity Method in Basins

In basins, ponds, tanks, etc. the Salt-Dilution method is not used, since here the primary purpose is not to determine the rate of flow but, knowing flow, to determine the detention period and thus the volumetric efficiency and other characteristics of the basins. The Salt-Dilution test cannot be used for this purpose because it does not give such a result. Thus, it lies with the Salt-Velocity test to serve for this type of work.

The method of use is the same as for conduit testing. However, the time observed is used to compute the detention period of the tank and not the rate of flow. The earliest recorded use of the test for this purpose was at the Lawrence Experiment Station of the Massachusetts Department of Public Health in model sedimentation tank tests. This is recorded by G. T. Fuller in a discussion of Groat's paper where he mentions the density effect and short circuiting due to density effects of the salt cloud. He noted some salt at the outlet almost immediately. This may be due to, incidentally, other than density effects, as will be discussed later. C. H. Capen, Jr. (6) also described the Lawrence tests that took place on filters before 1907 and on settling tanks in 1917. In his tests Capen used a salt slug as before and used the computed center of gravity of the curve as the measure of the flow-through period figuring the tank efficiency as $\frac{\text{Mean Time}}{\text{Volume}}$ / $\frac{\text{Volume}}{\text{Rate of Flow}}$. The average volumetric efficiency of all the tanks tested by him was 22.8%, ranging from 7% to 48%. Reference will be made to this paper again.

Very few other results have been published, since work in sewage is rendered nearly impossible by the highly variable chlorides; in water treatment only models are usually possible because of the danger of spoiling the water for use. The cost of such a test involving large amounts of salt is also a detriment. The main disadvantage, however, lies in the density effects that make most tests give a poor representation of the actual tank efficiency.

Dyes have been developed that can replace the salt, and more accurate results can be obtained with them in water tanks and conduits. In sewage an analysis of dye present is highly inaccurate because of color already present. Density is also a factor with dye usage.

Almost all flow in settling tanks etc. is laminar or streamlined. For that reason density effects are further emphasized.

The use of radioactive materials has been attempted before in two instances at least. In 1943 V. F. Hess (7) reported the use of radium as a tracer for flow measurement. He used the Salt-Dilution method as developed by Groat (4), and his results with large flows were excellent. The chemical difficulties were, however, enormous and practically it was not successful. Each test cost $5,000 and was very time consuming. In 1946 (8) use was made of the same Salt-Dilution technique to measure condenser water flow by Karrer, Cowie, and Betz in the plant of the Consolidated Gas, Electric Light, and Power Co. of Baltimore. The flow in this case was approximately 549 gallons per second. Radioactive sodium was used as the tracer, and the results were extremely good. As far as is known no one has used the Salt-Velocity Method with radioisotopes, and these are the only two known instances involving the Salt-Dilution method.

The limitations of the chemical methods in all the main fields of use are evident. In turbine testing and other high velocity conduits these tests give results that are accurate and useful. In other fields results are very poor. Into some of these gaps then, it is hoped, the utility of the radioactive tracer as a tool will be shown.

It should be noted that theoretical aspects of the Salt Flow tests have not been developed beyond an elementary stage and have not been of much use practically. Some new theoretical considerations will be presented that it is believed will contribute to a better understanding of the test.

Atomic Physics

The basis of all this new concept and usage of the atom was the discovery that the atom itself can be even further subdivided into different particles so small that the atom is huge in comparison. In the center of the atom is a nucleus around which revolves in certain fixed orbits infinitely small particles of negative electricity known as electrons. The nucleus of the atom is somewhat larger than the electrons and is composed of combinations of neutrons and protons, depending on the element. For a stable condition, the number of protons, which have a positive charge, must equal the number of electrons, which have a negative charge, the number of each being known as the atomic number. This number determines the chemical and

physical properties of the elements. The number of neutrons (protons with no charge) in the nucleus plus the number of protons is usually constant for a given element and is known as the mass number. However, by various means, either natural or artificial, the number of neutrons in the nucleus, and thus the mass number, can be changed. However this is done, either occurring in nature or by artificial bombardment, the atom with a changed mass number is known as an isotope. The number of protons and electrons is not changed, so that the chemical properties are not affected, and the atomic number is also constant. Because the element is not now in its normal stable state, it must become radioactive, that is, give off radiations, to try to bring itself back into balance. When this balance is effected, then the radiations cease, and the element has been changed to a new element of a different atomic number. This was always the dream of the alchemists, and now it has come true.

The radiations given off from these radioactive isotopes are of three distinct types. One of these is known as an alpha ray. This ray is composed of small alpha particles which are nothing more than the nuclei of helium atoms. This is not a high speed ray (one-twentieth of the speed of light), and it is not penetrating, being completely stopped by 3 to 8 cm. of air at standard temperature and pressure. However, this ray has a great deal of energy or ionizing power, which is absorbed by whatever it does strike. Generally alpha rays are given off only by naturally radioactive elements such as radium and uranium. Because of this and their low penetrating power and high energy they are not of any help in our problem.

A second type of ray is known as the beta ray. It is a stream of electrons travelling at high speed (approaching the speed of light). These rays are more penetrating than the alpha particles but have a smaller ionizing power. They are stopped, for example, by about 10 grams per square centimeter of aluminum, the exact amount depending somewhat on their initial energy.

The third type of ray is the gamma ray. It is an energy ray having no distinct particles and is similar to a ray of light. It is the same as an X-ray except that it has a shorter wave length. Today, in fact, with the larger, more powerful X-ray machines, there is some overlapping of ranges. The gamma ray is more penetrating, theoretically never being completely stopped, but generally about half will be stopped by 1.5 cm. of lead. Its ionizing power is similar to

that of a beta ray. The value of gamma rays lies in their great penetrating power, which enables them to go easily through water, pipe walls, etc., makes readings much easier, and in fact gives this experiment many of its main advantages. As mentioned before, beta rays are not so penetrating as gamma rays, but they are more frequent, and they are more efficiently detected. For these reasons they are very useful as an aid to the gammas.

Beta and gamma rays are the types usually found in synthetic isotopes that are produced in today's atomic piles and cyclotrons.

Isotope radioactivity is measured by the rate of disintegrations or atomic breakdowns per second. The common measure of this is the millicurie, equivalent to 3.7×10^{-7} disintegrations per second. Since only a few hundred disintegrations per minute are required with modern counting devices for a precise measurement of the intensity of radiation, it is evident that individual tests need involve only a minute part of a millicurie of radioactivity. It would generally be incorrect to use the term disintegrations, as only a small fraction of the disintegrations are recorded by one instrument and the percentage missed is not generally known. For this reason as well as others it is necessary to calibrate counting instruments with sources of known radioactivity.

Health Safety

The main drawbacks to the use of these new materials is the hazard involved to the personnel handling them. Overexposure to any of these rays could easily be fatal. Alpha rays are the most hazardous because of their great ionizing power, but their low penetration power makes them less dangerous except when inhaled or ingested, in which case they become extremely deadly. Beta rays are harmful to the skin and other exposed tissue. Gamma rays are very dangerous because of their penetrating power, which enables them to attack organs within the body that are safe from the other rays.

The results of overexposure are the same regardless of the type of ray, being dependent solely on the total ionizing power or energy of the rays received. Cancer of the skin, bone, etc., is a possible final effect. Preliminary warnings are a change in the white blood count, general fatigue, skin erythema, and many other symptoms. Genetic effects are also possible and may be serious. The health aspects are too varied and difficult to be explained here. However, it is to be

stressed here that the rays are dangerous and safety precautions are necessary and important.

A description of the safety measures that were taken may indicate the danger of the work involved.

As mentioned previously, a change in the white blood count or an anemic condition is one of the first indications of overexposure to rays. Because this is true, one of the tests to determine overexposure is the blood count. To establish the average value of the blood count, three tests are made for three consecutive months, then every three months another test is made, and any change is immediately apparent as a danger signal, and appropriate action is taken to prevent harm to the individual.

For further protection the worker wears continually a small piece of X-ray film. As gamma rays are similar to X-rays, they will expose X-ray type film. Beta rays also do this. Thus the amount of radiation received is measured. These badges are worn for a one week period, at the end of which time they are developed and compared with film exposed to known concentrations to determine the reading. If it is above the allowable, then work must be stopped and efforts made to remedy the situation.

Also used for health safety is a small ionization chamber, fountain pen size. This is essentially a condenser that is gradually discharged by the ionizations produced by the rays received. The amount of the charge removed from the condenser can be read on a scale and hence the amount of radiations received can be immediately read and work stopped when the daily limit has been reached. The chamber can be charged daily or more often by a small charging unit, and the scale brought back to zero to start recording again. The chamber is clipped to the pocket or some convenient spot on the body.

To monitor the working area, tools, and other equipment for possible contamination a larger battery-powered ionization chamber called a "cutie-pie" is used. It is a hand-operated instrument that when pointed at the contamination will read the strength of the radiations.

All handling of isotopes prior to dilution was by long tongs and other tools with extra long handles. Rubber gloves were worn by the worker to prevent contamination in cuts and possible burns from beta rays. Remote pipettes were used to prevent inhalation of the radioactive solutions. Shields were employed to prevent undue exposure.

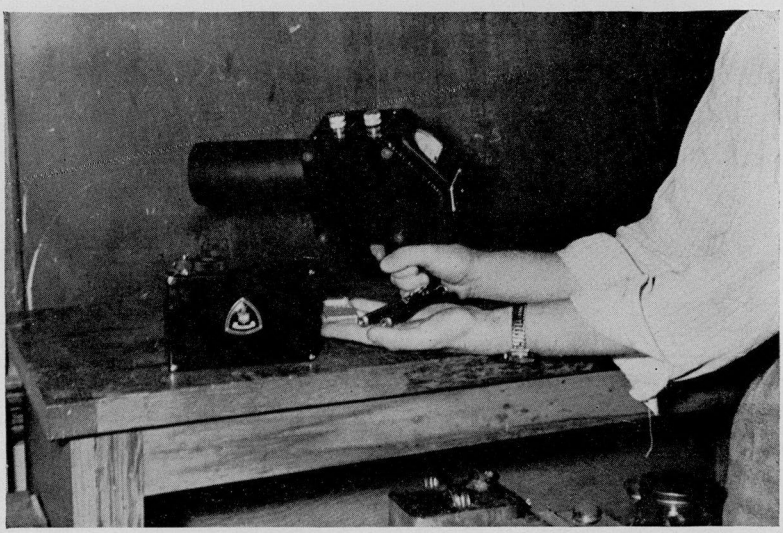


FIG. 1.—HEALTH AND SURVEY INSTRUMENTS. SURVEY METER OR CUTIE-PIE IN RIGHT HAND, FILM BADGE AND POCKET RADIATION METERS IN LEFT HAND.

Care must also be taken concerning the disposal of these isotopes. However, this was not too important in the work to be described, and it is too complex a subject to be considered here. Other safety features were practiced, but they are not of sufficient interest to be included in this paper.

SELECTION OF ISOTOPES

From the United States Atomic Energy Commission one may obtain radioactive isotopes of nearly every element. The selection of the proper and most useful one is of the utmost importance. Perhaps the most important factor to be considered in the selection is the isotopic property known as the half-life. This is defined as the time required for the strength of the radiations to be decreased by one-half, or the time when $R/R_o = 0.5$. This time varies widely among the various elements. For example, among the common radioisotopes sodium-24 has a half-life of 14.8 hours while chlorine has a half-life of ten million years. There are others with even shorter and longer periods. The importance of this value can be seen, for example, in the payment for isotopes which is on the basis of the strength when shipped, and if the shipping time is long, ma-

terial with a short half-mile is wasted. On the other hand, long half-lives are dangerous because the radioactivity, if spilled or ingested etc., will not decay within a reasonable length of time.

Another property of isotopes to be considered is the type of radiation. This is of great importance in detection and usage. Some tracers emit only beta rays and others both beta and gamma rays. In the tests described herein it was desirable to use an isotope having a gamma radiation, since this permitted detection through pipe wells.

The isotope finally selected for use was Iodine-131. The 131 refers to the mass number, which for non-radioactive iodine is 127. This element has a half-life of 8.0 days, and thus can be shipped by railway express; it is not so long-lived as to be excessively dangerous. Radiations from Iodine-131 spilled accidentally will practically stop after about 200 days. It has a strong beta ray (0.6 Mev.) and a gamma ray (0.367 and 0.080 Mev.). It can be obtained carrier-free; that is all the iodine present is radioactive with no other radioactive elements present. The price is reasonable (currently $1.00 per millicurie). It is shipped in a weak basic or a neutral solution as NaI. The amount of iodide actually present is very small being only 1.0×10^{-7} grams per millicurie. Radioiodine is well adapted chemically and physically to such uses as were made of it; it is not easily precipitated out of solution and is not readily combined or absorbed with other chemicals that may be present in water or sewage. Tests indicated that absorption by sludge and other solids was negligible.

Instrumentation

To detect the rays given off from these isotopes one needs instruments that operate a device known as a Geiger-Mueller Tube. This tube can be roughly described as a cylinder that acts as a cathode and a center wire that acts as an anode. As a ray enters the sealed space between them it causes the gas in the tube to ionize, which causes current to flow between the cathode and the anode for a short instant, which in turn snds an electrical pulse to the recording instrument. The tubes used in this experiment had mica windows in the ends, permitting the entrance of both beta and gamma rays.

The pulse sent from the tube can be recorded in two ways. The more common method is that of simply adding up the total counts or pulses received over a period of time. This is done by instruments

called scalers, and generally is accomplished by electrical circuits that record counts in the binary system of numbers.

The time of counting is obtained from an attached electric clock, which may be started and shut off in synchronism with the instrument. The pulses are recorded with almost no lag time. An instrument of this type is essential for working with samples of low concentration.

The second type of recorder is known as the counting-rate-meter. Instead of registering the total count received over a period

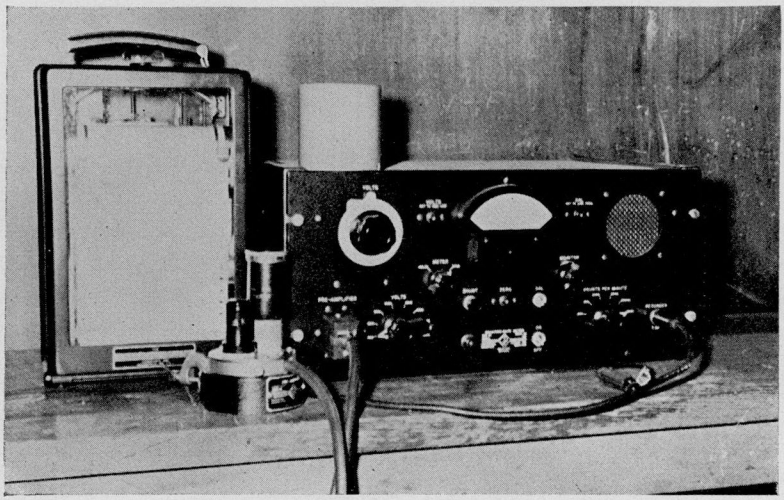


FIG. 2.—COUNTING-RATE-METER ON RIGHT, RECORDING MILLIAMETER ON LEFT, GEIGER TUBE BETWEEN.

of time, this instrument records the instantaneous rate of pulse generation in terms of counts per minute. A recording milliameter may be used in conjunction with this instrument. This records the output of the counting-rate-meter in ink on a moving paper chart. The rate of chart movement may be adjusted to various values. Thus the record of rate of counts being received by the counting-rate-meter is permanently recorded for future study. The basic principle of operation of the counting-rate-meter is as follows: a charge is built up on a condenser by the pulses from the tube; this charge leaks off through the resistance at a rate proportional to the charge. The reading of the charge on the condenser is proportional to the rate of

counts per minute. If the counts are increasing, the charge on the condenser is increasing faster than it is leaking off and hence, the reading increases. If the counts per minute are decreasing the charge is leaking off faster than it is being built up, and the reading decreases.

From a constant radioactive source the record will show a constant value except for statistical fluctuations. In comparing fairly long time intervals, say hours, the count rate from a given source appears to be constant, but in comparing short time intervals, say minutes, minor deviations are observed from the average rate; the disintegration rate of individual atoms proceeds in an erratic, non-uniform way, but approaches a constant rate when averaged over a sufficient length of time. Because of this there is one disadvantageous feature of this instrument. Since the statistical fluctuations of activity from a radioactive source are much greater when averaged over short periods than over longer ones, it is necessary not to record instantaneous rate values that are extremely variable, but to lengthen the period of rate determination so as to smooth out the variation more and more in order that the average can more easily be determined. This period of damping is controlled in this instrument by a resistance-capacitance (RC) time constant built into the circuit. As purchased, the instrument had an RC-value of between 20 and 60 seconds depending on the scale selected. It can be shown that the per cent fluctuation about the mean for this instrument by Poisson's Law is $\frac{143.1}{\sqrt{NRC}}$, where the N is the average counting rate in counts per minute. For a value of $N = 200$, for example, and $RC = 1$ minute, the per cent fluctuation is 10.1%, or 20 counts plus or minus. However, with the time constant (RC) of this order there is also considerable lag time in the response of the instrument. That is, when a constant source of radioactivity is quickly placed near the tube, the instrument will take some time to reach equilibrium or the actual reading desired. This time lag may be computed to be $t_{lag} = RC(1.15 \log 2NRC + 0.394)$, which for N equal to 100 counts per minute on the lower scales ($RC = 1$) is equal to three minutes.

As will be shown subsequently, this time was much too slow for some purposes. In testing flow in pipes, for example, it is necessary that the instrument-response to changes in tracer concentration be nearly instantaneous. In consultation with the manufacturer it

was decided to change the value of the capacitance in one of the circuits so that the lag would be reduced and yet so that the per cent variation would not be objectionably large. The effects of varying capacitance are marked. Many tests of lag, together with rate of rise or fall etc., indicated that a condenser of 0.5 mf inserted in the proper location was suitable for most work and one of 0.25 mf was also suitable for work where the tracer concentration changes were very rapid.

The value of RC with the 0.5 mf condenser was in the range of from four to twelve seconds. With this, the per cent fluctuation in counting rate increased. For example, when N was 200 counts per minute, the per cent fluctuation became 22.7%, or plus or minus 45 counts per minute. The disadvantages of the larger fluctuation were offset by the reduction in lag time, which with the capacitance of 0.5 mf became 0.45 minutes. These compromises had to be made and they proved to be entirely satisfactory, as will be shown. The value of the counting-rate-meter will become evident as the paper progresses and it is highly recommended for work of this type. Newer instruments are now available in which the condenser can be changed directly in the instrument itself, and this will prove to be of advantage to users.

One fact that has not yet been mentioned should be described briefly at this point. Counts will be received by the Geiger tube even though no radioactive source is near the tube. This is known as the background count. Its magnitude depends on the surroundings; the radiation derives from cosmic rays and radioactive elements that are present in very small concentrations in many materials. Sunlight may increase the background count. It varies from hour to hour throughout the day, and there is also a daily, monthly, and yearly variation in the count. Some tubes are more sensitive and have higher backgrounds than others. For a tube simply running in an open room, the average background might be 50 cpm. This figure can be lowered somewhat to perhaps 30 cpm by enclosing the tube in a thick lead chield to cut down the cosmic rays, etc., but for a particular tube there is a lower limit beyond which one cannot go for practical reasons. In considering the count from a sample, the background is always subtracted to evaluate only the rays received from the sample. Otherwise samples of the same activity would not be comparable because of the variation in the background.

Sample Testing and Technique

In using models for testing purposes with radioactive tracers, it was seldom necessary to take samples at intervals in the flow times to obtain the results desired. With models there is an advantage in the use of the counting-rate-meter and recorder with the Geiger tube placed as close as possible to the point where the reading is desired. The instrument plots the curve and gives a continuous permanent record with a minimum of effort and a maximum of efficiency. Sampling, with model tests, is not only less efficient but is also very poor in that the removal of a sample from a small flow may cause a serious disruption in the pattern of flow. Sufficient radioactivity can generally be used without excessive expense or danger so that the recording of tracer concentration may be accomplished directly through the liquid and surrounding pipe walls.

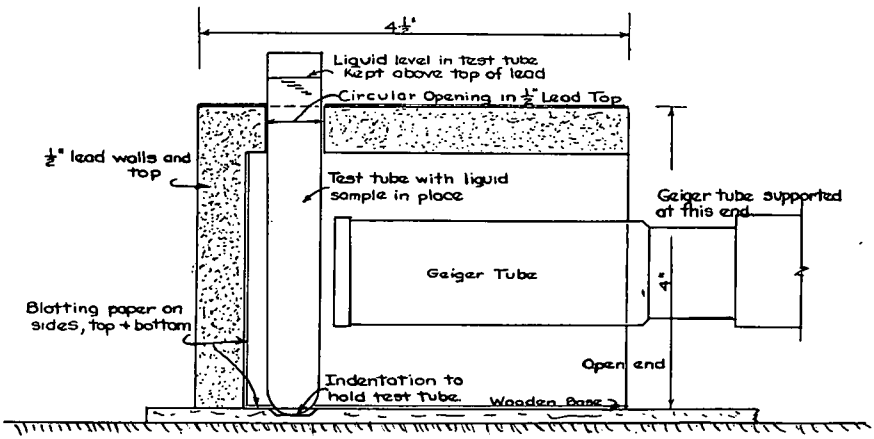
However, as the size of the pipes, tanks, etc., and also the volume of water and flow rate increase, as they do in field tests, the use of a Geiger tube recording through the pipe walls becomes both expensive and dangerous to the personnel. Actually it requires a comparatively high concentration of radioactive iodine to record through the walls of a pipe. With one-inch brass pipes used in the model tests, the concentration required in a static test with the face of the Geiger tube 4-mm from the pipe was a minimum of approximately 5×10^{-7} millicuries per milliliter (mc/ml). This value varies somewhat with the type of material from which the pipe is made, the diameter of the pipe, and other factors. At this concentration one cubic foot per second would require one millicurie per minute approximately, and where high flows of long periods are involved the cost is much too great.

Another disadvantage is the unfortunate fact that most field locations where measurements are desired are not suitable for the placing of a Geiger tube near the outside of the pipe. Unless the Geiger tube is well protected, it cannot be placed near open water because of the danger of splashing and vibration. Many locations such as manholes, ponds, most tanks, etc., are not suitable for field tests with instruments. The instruments do not appear to work well with portable generators that do not supply a nearly constant voltage source. Dipping counters are available that can be inserted in the liquid itself, but they too require fairly high concentrations of radio-

active material, as the glass shell cannot be too thin or it would be easily broken by changes in water temperature or pressure. This type would not be suitable in sewage and would be subject to contamination.

For all these reasons, then, in actual field testing it was necessary to take samples. As a radioactive material decays only according to its known die-away curve, and since with Iodine-131 this die-away is not rapid, it was generally not necessary to test directly in the field, but rather samples were collected and tested in the laboratory.

A drying technique wherein both types were counted proved extremely unreliable and thus having had so much difficulty with counting beta rays, it was decided to count gamma rays exclusively. As test tubes are commonly available, convenient and inexpensive, it was decided to count liquid samples in test tubes. As gamma rays easily penetrate water, there is no advantage to be gained from drying. For the purpose of this work the setup as shown in Figure 3



Cross-Section
TEST TUBE COUNTING SETUP
Not to Scale
Figure 3

was devised. The lead shield around the Geiger tube lowered and steadied the background count. The Geiger tube and shield were protected from spills by blotting paper. This setup proved to be quite satisfactory. The Geiger tube was placed as close to the sample as possible and locked in that position to insure reproducibility of

position. The tests performed to determine reproducibility were good and gave results well within the expected range of experimental error and did so consistently. Thus as far as reproducibility and ease of technique was concerned, this setup was excellent.

Tests were made with test tubes to see whether they were at all contaminated by the radioactive material. Counts were made with the test tube empty, full of radioactive solution, shaken dry, wiped dry, and washed and refilled with pure water. The results showed that the radioactive iodine does not contaminate the glass ware and is undetectable by this technique when simply shaken out of the tube. In this respect it might be well to note that some metal containers especially brass became contaminated with iodine and were not reusable even after extensive treatment with chemicals of various types.

Further tests were made to see whether the material had any tendency to be absorbed on the surface of the glassware, but it was found that no such tendency existed.

The test tube technique was excellent in all respects except one, and that was that sufficiently low concentrations were not detectable for all testing purposes. The lowest concentration detectable by this method was in the neighborhood of 5×10^{-8} mc/ml. As noted previously, when dealing with large flows this concentration would require large amounts of activity. The reason for this high concentration was that the beta rays, which are much more plentiful in I-131 and for which the Geiger tube is 100 times more efficient, were not being detected. New methods of testing then had to be devised to measure the low concentrations by making use of the beta rays.

The technique developed involved precipitation and filtering radio-iodide on a filter paper which when dried could be placed under the Geiger tube and counted. This is a well known technique in radioactive chemistry but was avoided at first due to the time required. It would be impossible to precipitate the small amounts of iodide present from a radioactive sample alone due to the solubility product effect. Actually one millicurie amounts to only about 1.0×10^{-7} grams and a dilution of 10^{-10} amounts to the amazingly small amount of only 1.0×10^{-17} grams of iodine. However, by adding a non-radioactive iodine solution of 0.01 Molar sodium-iodide and precipitating it with 0.1M silver-nitrate the major portion of the radioactive element will be precipitated. After allowing the floc to form for several minutes the solution was filtered through a Buckner funnel

under vacuum. The filter paper was allowed to dry on a glass plate. When dry the sample was then counted.

As before, when the drying technique was used, the reproducibility was not what would be desired. It was somewhat better than the drying method, however, and in view of the sensitivity attained was considered an improvement. Some erratic losses occur in removing the paper from the funnel when a small ring of material is left, in differing floc formation with various liquids, and the well known fact that precipitation techniques are never highly precise. However, very low concentrations (1.0×10^{-11} mc/ml) were easily detectable by this method and it is the only method found capable of doing this. Details of the method can probably be improved on, but it was selected for major use in the testing program and proved fairly satisfactory. Much work remains to be done in this field in the future.

Relation between Counting Rate and Tracer Concentration

One point of interest was the relation between the counts per minute obtained when using a given technique and the concentration of the radioactive source. In theory this relation could be established by estimating the number of distintegrations from the source—one millicurie equalling 3.7×10^6 disintegrations per second—that will register in the tube. This procedure does not give a reliable result since many of the variables, for example, the efficiency of the Geiger tube, are too indeterminate. The practical procedure is to measure the count rate from samples of known concentration. From such tests calibration curves may be prepared.

From the tests performed, it appears that over small ranges of concentration there is a linear variation of the concentration with count rate. Two examples shown here indicate this clearly. One, using test tubes, has a variation in concentration of about 10 to 1 and shows almost exact straight line relationship. The other curve from a large (2½ in. diameter) brass pipe shows the same effect (Figure 4).

However, over longer ranges of concentration (10^6 for example) the straight line relationship on arithmetic paper does not hold, and the concentrations do not vary directly with the counts per minute. As noted before, every technique and type of sample requires a different calibration curve; there is no way to interpolate from one

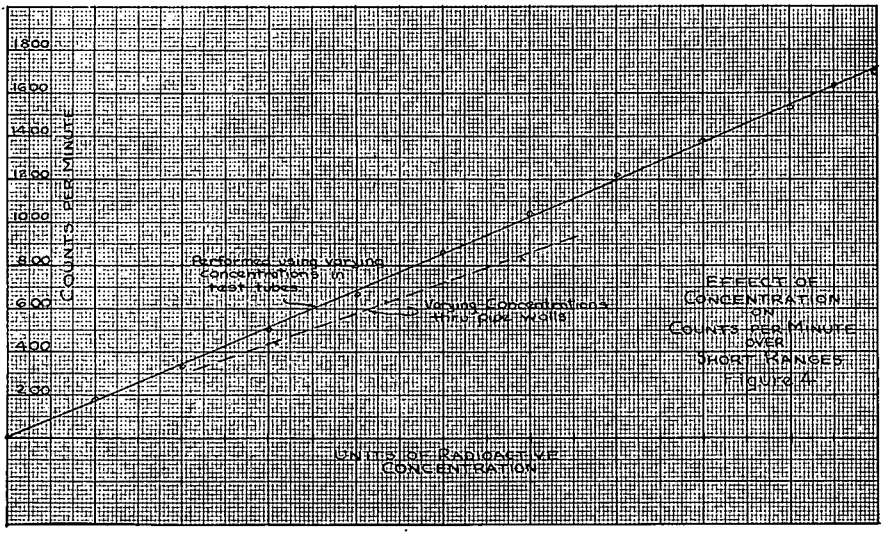


FIG. 4.

to another. A new calibration curve must be made for each test, if it is necessary that the tracer concentration be known precisely.

As an illustration, curves were fitted to the observed data found using the test tube technique and using the precipitation technique. The formulation from the test tube results is:

$$\text{cpm} = 3270(\text{conc.})^{0.2842}.$$

For the precipitation experiments this equation becomes:

$$\text{cpm} = 77{,}640(\text{conc.})^{0.3433}$$

where the concentration in both cases is in terms of millicuries per milliliter.

Thus it can be seen from the formulations that there are no parameters in common. The precipitation formula was obtained using results from water experiments. Other experiments using sewage are shown and these have their own slopes and intercepts showing that the type of solution used is also a factor. The equations, in themselves, are valueless except to illustrate the point that a new calibration must be made for each new factor if this relationship is desired.

The fact that over small concentration ranges a linear relation

exists between count rate and concentration greatly simplifies the calculations required to interpret salt flow test data. Indeed the linearity makes it unnecessary in these tests to evaluate the concentration at all. Plots of count rate versus time may be used in lieu of plots of concentration versus time in calculating nearly all pertinent information.

Because of this fact no percentage recovery of radioactive material in any of the tests was computed. Percentage recovery is defined as the relative amount of the original dose that can be accounted for by the sampling. This would have entailed a separate calibration for nearly every experiment, which effort would have been too costly in time and isotopes.

The equations serve to illustrate the range of concentration for which each technique is useful and give a quick, rough indication of what count rate can be obtained from a known concentration of radioactivity.

Probable Error in Counting

As the rate of counting from a given constant source varies exactly according to known laws of statistics it is often useful to apply these to the counted values in order to determine the magnitude of errors involved. As a measure of this discrepancy the term "probable error" is used. It is the error in the difference between the total count from a sample and the background count itself that can be expected to not be exceeded 50% of the time. It is defined by the formula:

$$\text{Probable error } (\%) = \text{P.E.} = \frac{67.5\sqrt{N+n}}{\sqrt{t}\,(N-n)}$$

where N = counts per minute including background.
n = counts per minute of background.
t = time of counting (minutes).
P.E. = probable error of (N—n).

This formula shows that as the time increases the error will decrease or simply that the true count of the sample is the limiting count as t approaches infinity. The probable error also decreases (in percentage) as the difference between N and n becomes larger, with t a constant.

This formula also illustrates why small concentrations are very

difficult to detect and count with accuracy. If N = 55 and n = 50 and it is desired to have a probable error of 10% then it would be necessary to count 191 minutes to get this rather unsatisfactory error. To obtain an error of 5% would take four times as long. Such times are generally out of the question so when the differences between N and n are small (about five) it is necessary that the counting time be extremely long if any accuracy is to be obtained.

To aid the experimenter in determining the correct counting times or concentrations required, the following chart was devised (Figure 5). Its coordinates are derived as follows.

$$\text{P.E. } (\%) = \frac{67.5\sqrt{N+n}}{\sqrt{t}(N-n)}$$

$$(\text{P.E.})(\sqrt{t}) = \frac{67.5\sqrt{n}\left(\sqrt{1+\frac{n}{N}}\right)}{N\left(1-\frac{n}{N}\right)}$$

$$(\text{P.E.})\sqrt{Nt} = \frac{67.5\left(\sqrt{1+\frac{n}{N}}\right)}{1-\frac{n}{N}}$$

The formula was computed and plotted on log-log paper as shown in Figure 5. It is in dimensionless form and so is useful for all counting tests.

Knowing the probable error desired, the time "t" required to count a given sample is easily obtained or vice versa. If a certain range of $\frac{n}{N}$ is being used constantly, the curve can be further expanded to make readings more accurate in the sections desired. The curve illustrates all the points previously mentioned as well as the fact that the probable error is not decreased substantially once a given value of time is reached, but that the decrease is quite rapid up

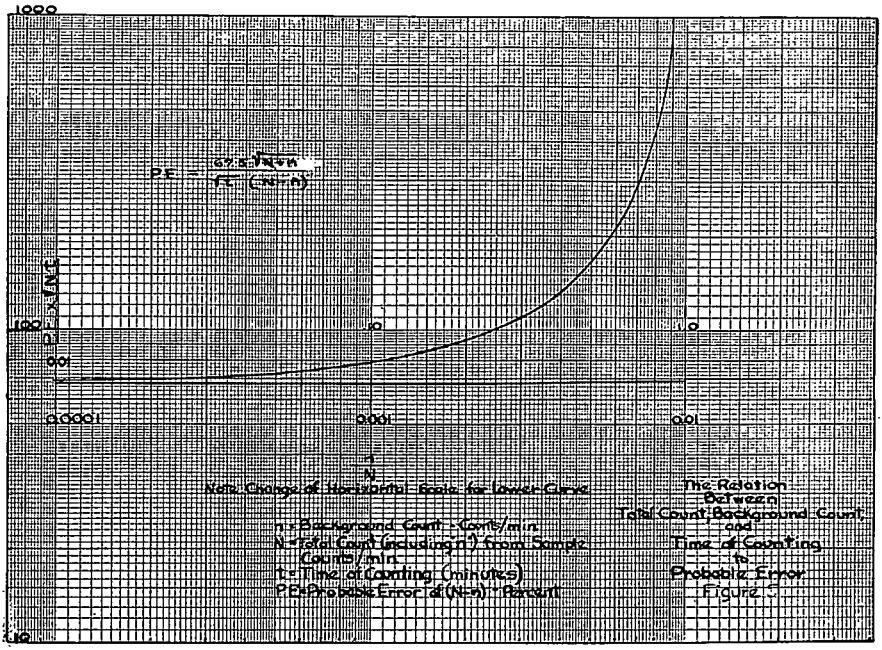


FIG. 5.

to that point. In other words for every sample there is an economical time beyond which the value of the probable error is not changed much. It also may be noted from Figure 5 that when the values of $\frac{n}{N}$ approach 0.01 the time of counting is not so important. For example, if N = 100, n = 50 and t is ten minutes the probable error is 5.24%. However, if t is 20 minutes the probable error is 3.7% a reduction probably not worth the additional effort, while if t is further increased to 100 minutes the probable error is still 1.65%. To illustrate the second point, if N was 500 and n was 50, with t equalling ten minutes the probable error would be 0.97% while if t was reduced 50% to five minutes the probable error has only risen to 1.37%.

The foregoing formulation provides a basis for experiments which must be designed to meet requirements of time, economy, and desired precision. It is important that all results be interpreted in light of the probable errors involved otherwise very erroneous conclusions may be made.

LABORATORY INVESTIGATIONS

The Salt-Velocity Method in Conduits

As the Allen method of measurement was well known and free from any apparent difficulties, it was decided that this type of test would be attempted in the laboratory to determine the feasibility of such experiments using radiotracers and to determine their limitations in such work. Also an attempt was made to throw some light on certain facets of the test which were somewhat indeterminate and obscure or to show how this could be done. There was no attempt to make very accurate tests for a direct comparison with the well known Allen test accuracy, as time was not available. The following results will show what was done, however.

Two laboratory setups were used. One involved a straight length of pipe about 85 feet long. The other, 105 feet in length, consisted of parallel sections connected with long radius 180° bends. In both instances the pipe was of the same diameter (1.21 inches). By the use of a constant head tank a constant flow could be maintained in the pipe (Figure 6). The flow rate was adjusted by varying the head. At the inlet end of the pipe a slug of radioactive solution (0.1 mc in 40 ml) was quickly injected under slight pressure. As the tracer cloud passed downstream and spread out its shape was recorded at different distances downstream by the Geiger tube operating the counting-rate-meter which in turn sent its signal to be re-

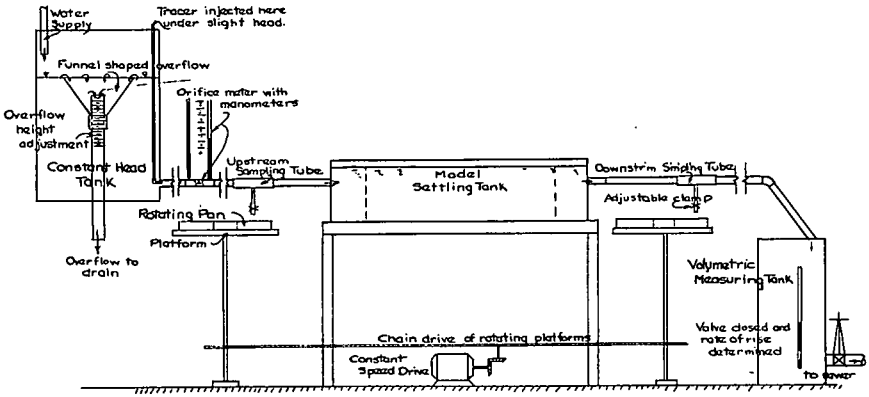


Fig. 6.—Schematic Layout of Tank Testing Equipment.

corded on the moving paper chart. The geiger tube was placed as close as practicable to the outside of the pipe.

The pertinent results of these experiments are shown in Table I. In this table the flow is in terms of gallons per second. The theo-

TABLE 1.—RESULTS OF SALT-VELOCITY TESTS ON LABORATORY PIPE

Section A

Constant Distance—Varying Flow (Distance = 94 feet)

Flow gps	Theo T Secs.	Mode %T	1st Trace %T	Av. Time %T	Ht. of Mode	Base Lgth. Secs.	Reynolds No.
0.0205	292.64	56.8	46.3	70.9	1.89	135	218
0.0841	71.71	82.0	73.6	88.6		19	9,410
0.1150	52.38	90.8	82.2	99.4	1.12	18	10,710
0.1367	44.27	98.4	89.4	100.0	0.89	12	14,210

Section B

Constant Flow—Varying Distance (Reynolds Number = 14,210)

Dist. ft.	Theo T Secs.	Mode %T	1st Trace %T	Av. Time %T	Ht. of Mode	Base Lgth. Secs.	Reynolds No.
21.4	10.25	103.8	73.1	139.0	1.36	—	14,210
36.5	17.06	95.4	73.3	101.2	——	—	14,210
60.9	28.32	92.9	82.2	108.2	1.28	—	14,210
75.6	33.99	93.9	82.5	100.0	1.17	—	14,210
94.0	44.27	98.4	89.4	100.0	0.89	—	14,210

Section C

Constant Flow—Varying Distance (Reynolds Number = 7,490)

31.3	27.8	56.2	45.9	78.6	——	—	7,490
44.8	39.4	69.1	55.4	80.1	——	—	7,490
53.4	46.9	72.1	63.0	86.0	——	—	7,490
78.2	68.7	77.1	69.9	85.2	——	—	7,490

Section D

Constant Flow—Varying Distance (Reynolds Number = 218)

21.4	69.35	57.7	39.0	84.7	5.30	64	218
36.5	116.5	60.1	38.6	67.6	2.80	67.5	218
60.9	195.52	70.3	38.3	60.4	——	86	218
75.6	219.44	54.8	44.2	57.9	1.06	—	218
94.0	292.64	56.8	46.3	70.9	——	135	218

retical time (T) was the time of passing, as computed using the velocity obtained by volumetric measurement of the flow rate. It is in terms of seconds. The mode (M) or point of maximum intensity is given in terms of a per cent of the theoretical time (T), as are the First Trace and the Average Time. The Average Time is defined as one-half the time between the first trace and the last trace of the cloud which is referred to as the Mean-time by Allen

(1). The height of the mode is given in terms of comparable units of no significant dimension. The base length is given in terms of seconds. The center of gravity for each curve was not computed because of the labor involved and the doubtful accuracy to be attained, as will be discussed later.

One of the purposes of the test was to attempt to find a parameter of the curve that would be suitable as a measure of the velocity of flow. Enough tests were not performed to do this accurately as that was not the main purpose of this paper. However, certain indications seem to point to several conclusions that can be loosely drawn.

As shown in Section B of the table, the Average Time, with high Reynolds Numbers, stands out as the most representative measure of velocity. (The 139.0% may be discarded as it is too close to the point of introduction to be accurate.) The Mode also gives certain indications that it too may be a constant percentage of the time. More testing is required to establish this. The First Trace of dye does not seem to be useful as it appears to vary percentagewise with both distance downstream and Reynolds Number.

With a decrease in Reynolds Number (Section A) a decrease is noted in all these parameters (percentagewise) and no consistent value is maintained. The decrease also does not appear to bear any direct relationship to the Reynolds Number. These tests indicate that at only high Reynolds Numbers where proper mixing does occur are the parameters valuable while below a critical value they cannot be used. This substantiates the experiments of Hooper (2) and others as discussed previously. Hooper found that good mixing occurred in a 2-inch pipe at a Reynolds Number of 2780. This is not exactly in line with these experiments which show that perfect mixing did not take place at an R of 7490, but there may be other factors involved. It would seem that additional testing along this line should uncover a relationship between these changes and some factor of Reynolds Number. From the tables it can also be noted that there is no significant change in these parameters with downstream distance that can definitely be identified. There are some indications of some variation, but further tests will be required to show the effect more clearly. This technique is admirably adapted to the making of such tests. Time did not allow the making of a sufficient number of tests here.

In Allen's paper (1), by way of comparison, the modal measure

was found to be quite accurate, and the initial appearance gave constant reproducibility. The Average-time gave very poor results. Whether these differences were due to the different technique or whether to different velocities, pipe sizes, etc., is not known. Again opportunity is presented for future work.

The base length increases with distance and decreases with Reynolds Number as would be expected. The modal height does not decrease with any noticeable relationship to length or Reynolds Number.

Thus, from this set of experimental data many inferences may be drawn. The indications are not substantial enough to produce definite statements but using this method such factors could be worked out with a reliable degree of accuracy. The experiments were easily and quickly performed with a minimum of expense ($0.17 per test). The results would be more accurate than those made with salt or dye due to the extreme sensitivity and the low concentrations that can be detected, that greatly improve the tail of the curve which by other methods is prematurely shortened.

No laboratory experiments were made with the Salt-Dilution Method.

Theoretical Investigations

In connection with this type of work considerable thought was given to the determination of the rapidity of spread of the dye cloud as it passes downstream under varying conditions, and the formulation of such spread.

A preliminary formulation has been evolved which while not entirely satisfactory gives answers fairly close to the actual result. Several sources of data were used for these computations. They vary widely as to pipe diameters, and there are many phases of each experiment that were unknown that may be a factor in the results. However, they are the best that could be obtained at the moment. The sources include tests made by Dr. J. E. McKee of the firm of Camp, Dresser, and McKee using dye in sewers; tests made in Louisville sewers by Johnson (9) using both salt and dye; the previously discussed pipe experiments performed with radiotracers; and two large aqueduct tests made using salt.

Despite the great variation in source data the results from these tests have been arranged satisfactorily and serve as a good indi-

cation of the results to be expected in the spread of the cloud as it goes along. It was immediately recognized that Reynolds Number must be an important factor, and it is one of the parameters used. The actual spread of the tracer in each case was measured as the time from the first trace to the last trace (Δt). The coefficient of variation of the curve (theoretically equal to the standard deviation divided by the mean) was approximated as $\frac{\Delta t}{6T}$, as $\frac{t}{6}$ is approximately the standard deviation and T is the mean time or the center of gravity of the cloud equal to the length traversed divided by actual velocity. The relationship, which was determined mostly by empirical methods, is shown as a plot of the coefficient of variation multiplied by the flow (cfs) versus the Reynolds Number. The values as plotted in Figure 7 show a fairly constant straight line relationship on the log-log paper. It is not perfect but there is a definite trend that can easily be seen. Results taken from a constant setup by a constant method of procedure should give much more accurate results and be of much greater value.

By means of pure theory the distribution of the tracer cloud as it passes downstream can be described. The formula for this has been worked out for streamlined flow in circular tubes. For turbulent flow and in other shapes the work is much more involved and has as yet been uncompleted although considerable thought has been put on the subject.

The velocity distribution across a circular tube is known to be equal to $V = V_m \left(1 - \frac{r^2}{R^2}\right)$ in streamlined flow, where V is the velocity at any point a distance r from the centerline, V_m the maximum velocity at the center and R the radius of the tube. As the shape of the velocity distribution curve must be the same as that of the tracer cloud it can be shown that the concentration (u) passing at any time (t), a point any distance (x) from the start is equal to:

$$u = \frac{1}{2V_m t \sqrt{1 - \frac{x}{tV_m}}}$$

where u is the average concentration across the pipe.

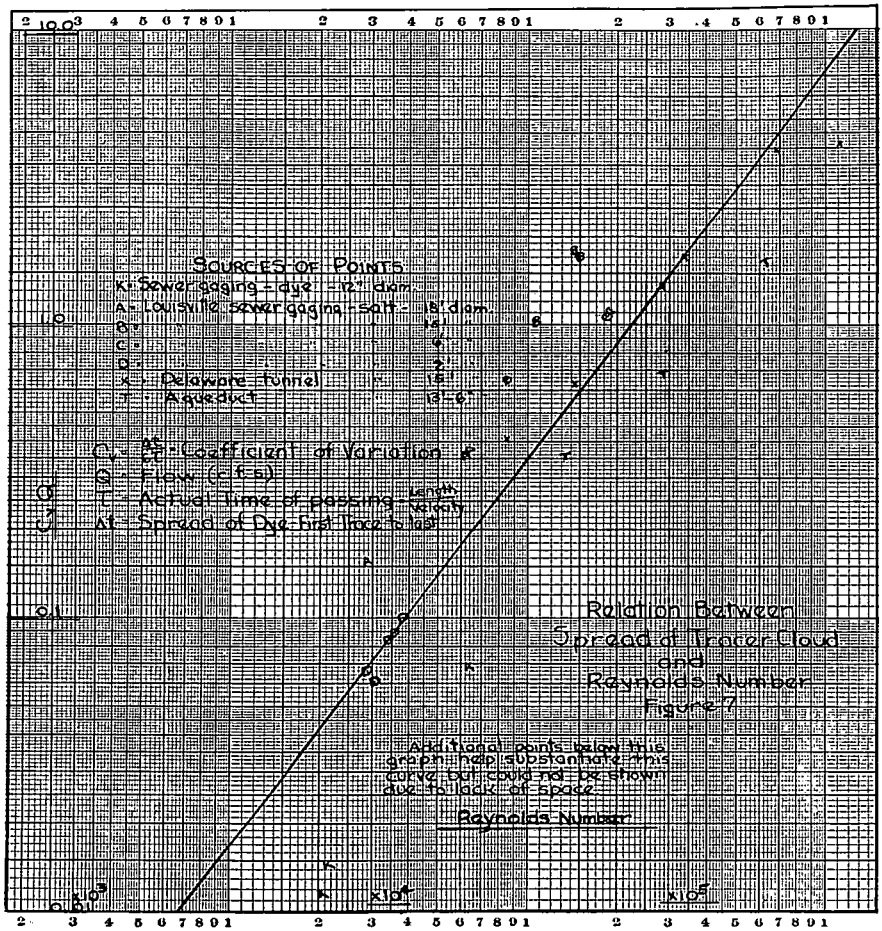


FIG. 7.

Operating further on this equation it can be shown that the first moment or the mean of the curve is equal to infinity. That is, although the curve has a definite area ($u_o = 1$) the tail of the curve is so long that the moment is infinity. Practically speaking, that indicates that there is always some of the tracer cloud remaining to go by. This is well illustrated in Figure 8 which shows that at the end of fifteen detention periods there still remains 1.7% of the total which has not passed and at the end of 100 detention periods 0.25% still remains. The tails of the curves are, therefore, of the utmost im-

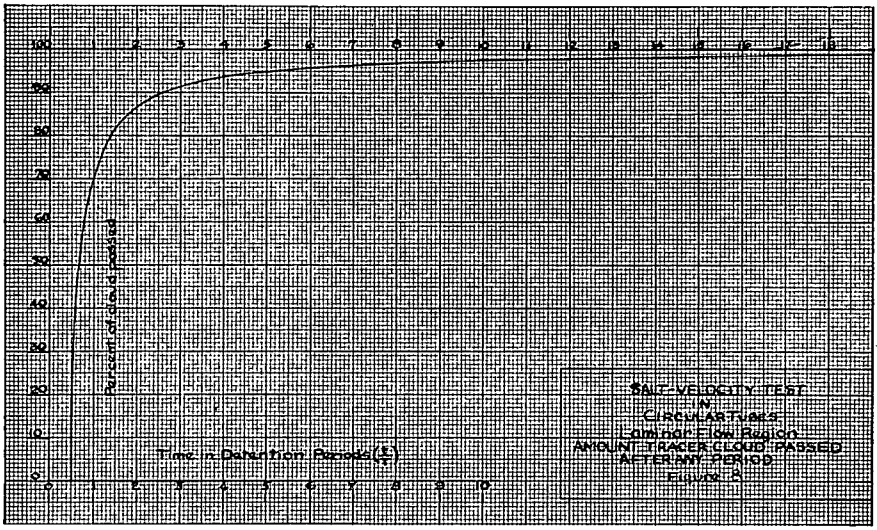


FIG. 8.

portance, and it is these that the radioactive tracer will improve upon most. From the same equation the median can be shown to equal 2/3 of the theoretical detention period (T), and the Harmonic mean equal to ¾ (T). In this streamlined flow in tubes the best measure of the detention period is the 70.7 percentile or that time in which 70.7% of the total curve has gone by.

No attempt has been made to check these equations experimentally as few tests were performed in the streamlined range and exacting work which this checking would require was not possible at this time. When the equations similar to these have been worked out for all conditions of flow the Salt-Velocity test will be much improved and theoretical work can then be applied to results obtained to give even greater efficiency. Meanwhile the equations for streamlined flow furnish many leads that had not been previously considered and may have considerable value in the future.

As no mathematically exact solution for the relationship between the spread of the tracer cloud was evident for the case of flow in channels such as rivers etc. an attempt was made to do this by other means. Included was the effect of varying the velocity in a vertical plane through the various depths of the river.

By means of a mathematical, semi-mechanical analysis a numeri-

cal model of the cloud as it spread out was prepared. Two conditions were investigated. One where the flow was uniform, that is a constant velocity both horizontally and vertically; and the second case where only the velocity in the horizontal plane was constant while in the vertical plane changes occurred at various strata. This model cannot, of course, be compared directly to flow patterns as naturally occur in water courses due to the numerical assumptions that had to be introduced, but the model does serve to indicate how the spreading takes place and some of the factors that are involved.

A model tracer cloud was set up similar to the following with five different layers being considered. The numbers in the spots indicate average tracer density over the area and the horizontal divisions are equal distances from the center. It is assumed that the view is of the vertical cross-section of the cloud in two-dimensional flow. The initial state of the cloud following injection is assumed to be as follows:

Distance: (Units of time)	—2	—1	0	1	2
Strata #1	1	2	4	2	1
Strata #2	1	2	4	2	1
Strata #3	1	2	4	2	1
Strata #4	1	2	4	2	1
Strata #5	1	2	4	2	1

In the case of uniform flow all portions of the cloud move uniformly in relation to one another so only movement of the whole need be considered and that is unimportant to this problem so the cloud may be considered as motionless. However, diffusion from higher density regions to those of lower density does take place and for the purposes of the analysis this diffusion was assumed to amount to 0.2 of the difference in concentration taking place in a time unit of t. Thus, at the end of time t the cloud in uniform flow would have assumed the following shape:

Distance: (Units of time)	—3	—2	—1	0	1	2	3
Strata #1	0.2	1.0	2.2	3.2	2.2	1.0	0.2
Strata #2	0.2	1.0	2.2	3.2	2.2	1.0	0.2
Strata #3	0.2	1.0	2.2	3.2	2.2	1.0	0.2
Strata #4	0.2	1.0	2.2	3.2	2.2	1.0	0.2
Strata #5	0.2	1.0	2.2	3.2	2.2	1.0	0.2

Similarly the steps can be carried out to any number of units of time t.

For non-uniform flow a flow pattern had to be arbitrarily selected. For the example shown here the following pattern was used.

Strata #	Velocity: (Units per time t) All in a direction toward the right.
1	2
2	3
3	2
4	2
5	1

This distribution is somewhat similar to the usual velocity curve found in such two dimensional flow in rivers etc. Thus, in each time interval the various strata move ahead at the given rate, or in relation to one another the first, third, and fourth strata remain still, and the second goes ahead one unit while the fifth drops back one unit. Starting with the same model as that described for uniform flow, with the velocity distribution as shown, and with diffusion now taking place in two directions according to the same scheme as before, at the end of two steps or two t time units the cloud is as follows:

Distance: (Units of time)

—6	—5	—4	—3	—2	—1	0	1	2	3	4	5	6
		.04	.28	.92	1.92	2.60	2.36	1.32	.48	.08		
				.12	.64	1.56	2.56	2.40	1.68	.76	.24	.04
		.04	.28	.92	1.92	2.60	2.36	1.32	.48	.08		
		.08	.48	1.32	2.36	2.60	1.92	.92	.28	.04		
.04	.28	.92	1.92	2.60	2.36	1.32	.48	.08				

Strata are in the order of #1, #2, #3, #4, and #5.

This process can also be carried on for any number of steps each one of which represents the changes occurring in a unit distance or time.

The shape of these curves having been determined, the various measures of the curves at each step were computed and can be compared in the following table. Table II.

The standard deviation is a measure of the spread or width of the curve and is the square root of the average distance from the mean squared. The average distance from the mean is the average Deviation. Geary's ratio is a measure of the normality of the curve and is equal to the Average Deviation divided by the standard deviation. The normal value for this ratio is 0.7979.

An examination of the data reveals some interesting results. It is readily seen that the standard deviation of the non-uniform curve

TABLE 2.—VARIATION OF PARAMETERS WITH CLOUD SPREAD

A. Uniform Flow—Both Horizontal and Vertical Velocities Constant.

Step	Std. Dev. (s)	Change in s	Change in S^2	Aver. Dev. A.D.	Geary's Ratio	Mode Ht.	Mode x s
I	1.095			.80	.7305	20	21.90
		.170	.4				
II	1.265			.96	.7588	16	20.25
		.149	.4				
III	1.414			1.184	.7694	14	19.80
		.135	.4				
IV	1.549			1.200	.7747	12.7	19.65
		.124	.4				
V	1.673			1.300	.7771	11.75	19.70
		.116	.4				
VI	1.789			1.396	.7803	11.00	19.70
B. Non-Uniform Flow—Velocity in Horizontal Plane Constant Vertical as Shown on Page 64.							
I	1.095			.80	.7305	20	21.90
		.319	.800				
II	1.414			1.088	.7694	14.0	19.80
		.419	1.360				
III	1.833			1.432	.7812	10.7	19.65
		.441	1.808				
IV	2.274			1.791	.7878	8.67	19.70
		.438	2.189				
V	2.712			2.151	.7932	7.29	20.45
		.431	2.521				
VI	3.143			2.506	.7972	6.46	20.37

increases much more rapidly than that for the uniform, that is, the cloud is spreading out much faster. This is what would be expected with the increased mixing effect. The same can be seen from the average deviation. Geary's Ratio shows that the non-uniform flow curve is becoming normal at a much faster rate than is the uniform. At the end of six steps it is nearly perfectly normal and from Step III on is nearly so. The change in the standard deviation squared (often called the variance) is constant for uniform flow and is increasing steadily for the non-uniform case. The change in the standard deviation is fairly constant for non-uniform flow and is decreasing with time for the uniform test.

Now the standard deviation being a good measure of the width of the curve and the Mode equal to the height, the product should give a good idea of the area of the curve. As the area of the curve must be constant this product must also be fairly constant and for non-uniform flow this is seen to be true. The product of the mode

and the standard deviation in Step I is 21.9 and in Step VI it is 20.4. It is also fairly constant for uniform flow. In Mason's paper (3) he states that the mode should decrease with the square root of the length traversed. If this is the case, to maintain the area or the product constant the standard deviation must increase with the square root of the distance. This, it definitely does not do in this example and in others computed, so grave doubts must be cast on the theory of Mason's. The modal values do not decrease in this ratio either to further substantiate this surmise. Modal heights two units apart should be in the ratio of the square root of 2 or 1.414, and this is seen not to be the case. The rule is not followed in other ranges either. No attempt will be made to say how this spread may be formulated as intricate mathematics as yet unsolved are involved. It does appear that the spread is increased by a nearly equal amount over equal distances in this type of two-dimensional flow. Further work must be done to learn more about such points and these model analyses may be helpful in casting more light on the intricate subject.

Field Investigations

When the testing program began it was soon discovered that the testing of large conduits, such as the penstocks of power plants, as Allen did, would be out of the question at this time. The reasons for this have been discussed in preceding chapters and are now probably obvious. Mainly, in order to record the radiations through the pipe walls a concentration of radioactivity would be required that would make the test too dangerous and costly where such high flows are encountered. If sampling then is required, some method must be worked out for taking a representative sample and this is probably more difficult than placing electrodes for the Allen test. With the high velocities encountered in these conduits such samples would not likely give a representative curve, as they could not be taken with sufficient rapidity. Thus, as yet, with this method there is no replacement of the Allen test in large high velocity conduits. Possibly in the future when more time can be spent on the problem a solution may be found, but none will be attempted here.

With the lowered velocities and flows found in smaller pipes (less than about 36 inches) such as those particularly encountered in sewage work, this method of using radioactive tracers is very satisfactory. In smaller pipes (about 6 inches in diameter) sufficient

radioactivity can be used, depending of course on local conditions, health safety, and disposal problems, to record through pipe walls as was done in the laboratory tests. This method could be used to great advantage in water systems except for the danger involved if it were a potable supply. This direct reading method was not field tested as any test would be exactly similar to the laboratory work.

Tests were made, however, by means of sampling on a sewer to measure the velocity and to see if some light could be thrown on the spread of the tracer cloud as it passes downstream. The tests were made through the cooperation of Mr. Arthur Weston, Chief Sanitary Engineer of the Sanitary Engineering Department of the Massachusetts Department of Public Health who has shown tremendous interest in these tests and given a great deal of help. Through his office the sewer to be tested was selected and it proved to be ideal for such purposes.

The sewer was located between Rutland and Worcester, Mass., and was the property of the Metropolitan District Commission over whose water supply shed it passes. Of tight cast-iron construction which allowed no infiltration, it was perfect for such tests. Throughout the section tested it had a uniform diameter of 16 inches and a constant slope of 1.8 feet per thousand feet. Convenient manholes allowed easy sampling and at its lower end (Rutland-Worcester line) a Kennison nozzle gave a flow measurement, allowing checking of the test measurements.

Two series of experiments were made on this sewer. Again they were not made with any attempt at absolute accuracy as time and money was not available for elaborate equipment but mainly to show the possibility of the use of radiotracers for this work and to discover the properties of moving clouds. The first samples were tested with the previously described test-tube technique. The results of two runs gave an average reading of the flow as 0.23 MGD against a meter reading of 0.32 MGD. As far as can be determined, this error must be due to the inaccurate determination of depth. This was measured near the nozzle whose backwater may have affected the readings. A difference of one inch would give a remarkably good agreement. All other known sources of error would have thrown the difference in the other direction. Thus, the true accuracy of the first tests was not known.

The use of the Counting-Rate-Meter in the field was attempted

during this test, but the results were largely unsuccessful due to improper voltage regulation from a portable generator, improper operation of the instrument and the tube due to excessive dampness, and the poor location available for placing the Geiger tube in close proximity to the radioactive source. No other field tests with the instruments were undertaken due to the possibility of a combination of these and similar difficulties occurring.

A second series of tests was run and was also very successful. They were performed in the same sewer over a perfectly straight length with the same constant slope and diameter. Samples were taken every thirty seconds from two stations, one 561 ft. and the other 1,122.5 feet downstream from the dosing point. The average meter reading over the testing period was 0.40 MGD as closely as can be estimated from the graphical record which was to a very small scale with a very thick pen line. The computed value from the first station's samples was 0.435 MGD. This is only a 10% difference which is not excessive considering that the flow was varying during the sampling period and the possible errors involved in reading the graph and the errors that may have been introduced by improper cleaning and maintenance of the meter. From the downstream manhole the measurement gave 0.415 MGD and the time between the centers of gravity of the two curves from the two manholes gave 0.396 MGD, both remarkably good checks.

These tests have thus shown that the use of radioactive tracers is possible and reliable for work in measurements of the type described. In other types of work that were not field tested there is every indication that the method will be just as successful although modifications of the method described may have to be made. No means is available at the present time that is satisfactory for comparison with the radiotracers in the measurement of sewage flow. All other methods break down in such work; dyes are easily affected chemically; salt tracers are inaccurate as the chloride concentration in sewage varies constantly and with the large amounts required density effects are a strong possibility. Only 2.5 ml of radioactive solution was added in these sewer tests so density effects are improbable. Only about 3 millicuries were used in each test. The last series was tested by the precipitation technique and it proved to be more satisfactory than the test tube method. Accurate results are possible as has been clearly shown with more precise sampling methods the accuracy will be

further improved. Sewer gauging has thus become possible by accurate means and the new method will undoubtedly have many new uses in the future.

A field test was performed to attempt to gauge a small brook adjoining the Pondville State Hospital in Norfolk that flowed into the Stop River. Results of this test were not significant and this is explainable because of the sampling technique employed. Detectable traces of the radioactive slug were found but only irregularly. It was noted from a preliminary dye run that the tracer slug was considerably broken up in travel and came by the sampling point (177 ft. downstream) in small clouds very irregularly from backwater pockets along the bank. Thus, samples which were taken every 15 seconds gave only erratic uninterpretable results. In a larger stream the peripheral conditions would not be such a large factor and better results would be obtained. For small streams of this sort (about 3 cfs) a different sampling technique must be employed using either a continuous sample or some other way to get a representative sample. About 3 millicuries were required for this test to give detectable readings 177 ft. below the sampling point.

At the same time as the first set of Salt-Velocity tests were run, a Salt-Dilution test was made in the same sewer under the same conditions. A strong solution of radioactive iodine of known concentration was allowed to fall into the main flow at a known rate (146 cc/min) which was maintained by a constant head bottle. Samples were taken downstream and by methods described previously the flow was measured. The results of this test gave the flow as 0.49 MGD against a meter reading of 0.54 MGD. The results are satisfactory considering the possible errors in the meter and depth measurement. Also the test tube method of testing used here does not give as good efficiency as the precipitation method and the use of precipitation would undoubtedly increase the efficiency but the method had not been developed at the time. This is the only way that radioactive tracers have ever been used in the past for liquid measurement and very good efficiency was reported in all cases so it is perhaps unfortunate that more tests of this type were not performed at this time but money, time, and permission were not available to do others. The Salt-Dilution method may be even more convenient and accurate than the Salt-Velocity methods in large conduits especially with the radioactive tracers. The use of this test with high flows using radio-

active tracers may be a strong possibility. All the disadvantages of the salt or dye have been removed and all the advantages retained.

In concluding this section, it has been shown that radiotracers can be used for making flow measurements in smaller conduits at least by both the Salt-Dilution and Salt-Velocity methods. Additional work will have to be done to establish all the sources of error and develop more accurate techniques. These tests have shown what can be done and the value of the radioactive tracer can be seen from the results.

The Salt-Velocity Test in Basins

From a sanitary engineering standpoint the use of the Salt-Velocity test in basins such as settling tanks is, perhaps, more important than its use in conduits. Very little work has been done in the field due, mainly, to the difficulties involved in using present methods, as described previously. To design settling tanks and other similar hydraulic structures the engineer must rely on empirical relationships developed from actual practice, but these often do not result in good design. Even the question of how to improve a poor tank cannot be efficiently answered. For these reasons the remainder of the paper will almost exclusively be devoted to problems concerned with flowing-through tanks and how the radioactive tracer may be used in developing tests and designs which will better serve the needs of such engineers.

Laboratory Tests

Testing with models has been proved an effective way to determine in advance whether a design is satisfactory or after construction and operation to determine how it may be improved. Therefore, the first work was concerned with tests made on a model sedimentation tank. The tank was constructed as shown in the photograph Figure 9. It was constructed of lucite with fluorescent lighting beneath so that the spread of dye through the tank could be noted. Baffles were used and were removable in various combinations to achieve various flow patterns. The water was supplied from a constant head tank as shown in Figure 6. The variable overflow weir allowed a variation in the rate of flow to be easily and accurately made and controlled over a wide range of flow. The flow was metered in two ways. The first was by means of an orifice meter constructed as shown in Figure 6. This operated satisfactorily after calibration except for

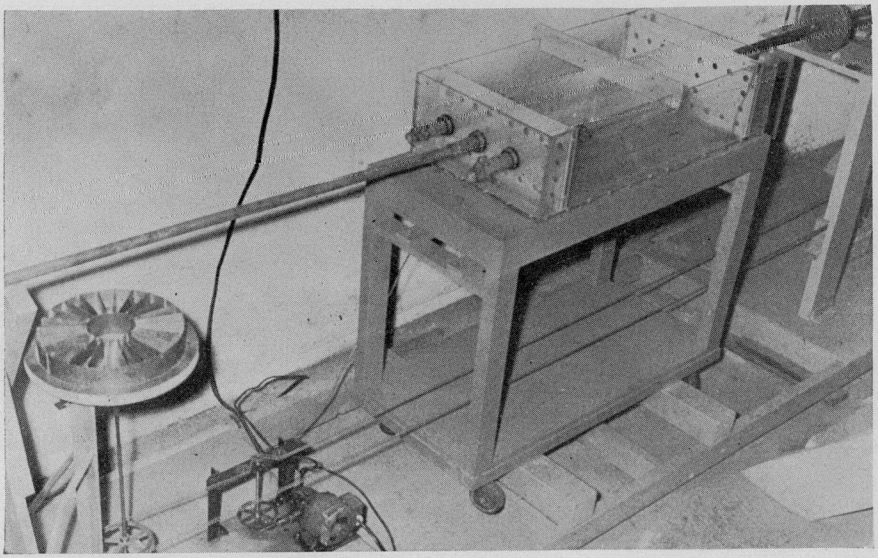


FIG. 9.—GENERAL VIEW OF TESTING SETUP.

some difficulty with air bubbles collecting in the meter. Some removal of air was effected by stirring the water in the constant head tank. However, this was not entirely effective so a second measurement was made volumetrically by noting the time required to fill a large cylinder between two known levels. This flow measurement was made before and after every test and generally little variation was noted. The depth of water in the lucite tank was variable with the applied head and therefore, depth readings were made at each end, and the results averaged to obtain the average depth. As the area was fixed the volume of the tank could thus be determined.

For laboratory work with water a dye is the best tool that has been developed up to this time. Therefore, to give a basis of comparison for the radiotracer, dye tests were first made on the tank. In this instance methylene blue was selected as being satisfactory. The slug of dye was injected into the flow just inside the pipe leading from the constant head tank. To inject it, a glass tube led up to a supply tank several feet above the water surface to give a head to force a quick injection. When the valve on this line was opened a known amount of methylene blue (0.5 gms per liter) solution (40 ml) would run into the flowing pipe in approximately three seconds.

Many tests were made with longer injection periods, but these served not only to lengthen the sampling period unduly but also seemed to introduce other variable factors while adding nothing to the betterment of the results.

As this was a one man test an arrangement was developed to take samples from the center of the pipe at each end of the tank in a continuous manner. A sampling tube (Figure 10) was inserted

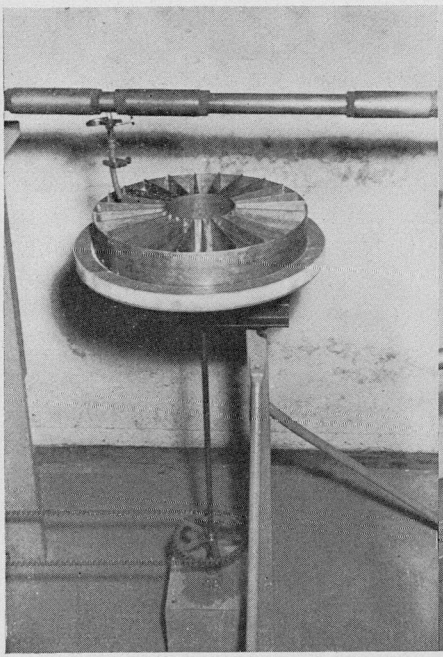


FIG. 10.—SAMPLING TUBE AND PAN.

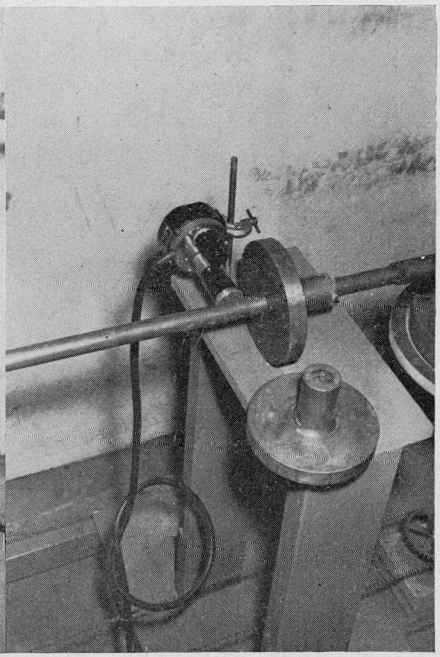


FIG. 11.—GEIGER TUBE IN PLACE BESIDE PIPE

just previous to and just after the lucite tank. From this a continuous sample from the center of flow would be drawn off. This sample flow, which only amounted to a small fraction of the total flow, was collected as it dropped from the sampler in rotating compartmented pans (Figure 10). These pans (one at each end) rotated in synchronism at a known constant rate, and thus a series of composite samples each over a peroid of about twenty seconds could be taken simultaneously at both ends of the tank, an impossibility otherwise for one person to do.

The samples could then be collected after the test was completed, transferred to appropriate containers, and the concentration of the dye determined. The concentration of dye was measured by a photoelectric device which compared the light transmittancy of the sample with that of pure water and gave a reading of this transmittancy as a per cent of that of the pure water. From this per cent reading, from curves prepared by calibration, the actual concentration of the dye in grams per cubic centimeter could be obtained. Thus, knowing the time of each sample and the concentration, the curve could be obtained for the influent and effluent conditions. After many tests it was found that samples at the influent and served no useful purpose as the dye distribution at that point was very concentrated and samples gave a poor indication of the curve. Thereafter, only the effluent end was sampled, and the initial time was measured as the time of introduction.

With the radioisotope tests that were made to show their effectiveness in comparison with the dye, the same procedure was followed in every respect so that both sets would be exactly comparable. However, with the isotope tracer no samples needed to be taken. The Geiger tube was placed next to the outer pipe wall at the same location as the sampling tube on the downstream side of the tank as shown in Figure 11. Thus, as the radiotracer passed directly in front of the tube the radiations were received and recorded by the Counting-Rate-Meter in conjunction with the recording milliameter. The curve was thus plotted and could be analyzed at leisure. The Geiger tube was shielded from stray radiations from sources such as the liquid in the lucite tank by heavy lead collars on both sides of the tube that fitted around the pipe (Figure 11). Actually tests showed that in straight lengths of pipe where there were no other sources of radioactivity the collars were not necessary, and in tests such as the pipe tests previously described they were not used. However, for these tank tests they were considered essential. The tube was placed as close as possible to the pipe walls as there is a rapid decrease in radiation intensity with distance.

Before describing the tests further it will be well to insert a brief description of the various measures employed to describe the results, i.e. the parameters or measures of the curves. Examples of the curves obtained are shown in Figure 12. The mean flow-through time (T) is defined as the average time for the dye or radiotracer

to pass through the tank or $\left(\frac{\Sigma \text{ conc. x time}}{\Sigma \text{ conc.}}\right)$. The mode (M) is the point of maximum tracer intensity. The median (X_{50}) is the time for the average particle to pass through or, in other words, fifty per cent of the particles pass in that time. The ten-percentile (X_{10}) is the time for the first ten per cent of the tracer to pass and the ninety-percentile (X_{90}) the time for ninety per cent to pass by. The Decile Range (D) is a measure of the spread of the tracer curve and is equal to $X_{90} - X_{10}$. The First Trace is the time of first appearance of the tracer. Per cent recovery is the per cent of the dye introduced that can be accounted for by the samples, or

$$\frac{(\Sigma(\text{conc. sample})\ (\text{time period of sample})\ (\text{flow}))}{(\text{Total Tracer Added})}.$$

It is used in the case of the dye only. Nearly all of these values are presented in terms of per cent of the theoretical detention period in order that they will all be comparable which would not be the case otherwise. Many of the curves are plotted and comparisons made on the basis of the cumulative concentration curve. The concentration values are simply expressed as a per cent of the total area and then summated with respect to time. That is, at any given time the curve represents the percentage of the total curve that has passed up to that time. These curves are a bit difficult to interpret at first, but they have many advantages that out-weigh the slight disadvantages. One main advantage is that the mean of the curve is represented by the area to the left of the curve.

Previously it was described how the Counting-Rate-Meter was calibrated and the time constant changed to suit conditions. Before going on to describe the advantages of the radiotracer over the dye, it will be convenient to describe the further calibration of the Counting-Rate-Meter for its use in this particular test. To do this the Scaler was brought into use. Its Geiger tube was placed on the pipe's opposite side from that of the rate-meter. Thus, they both received the same signals at the same time, and as the Scaler has no lag time any differences would be immediately noted. The Scaler does not record directly, but readings were taken every fifteen seconds without difficulty. The results from the Scaler are already cumulated

so for comparison a cumulative plot was made of the Rate Meter curves. A comparison is shown in Table 3. All of these experiments were performed at the same rate of flow, and therefore the detention period $\left(\frac{\text{Vol.}}{\text{Flow}}\right)$ will be constant. The values given are in

TABLE 3.—CALIBRATION OF COUNTING-RATE-METER WITH SCALER

Type	Mean or % Efficiency	Median or X_{50}	Decile Range
CRM—No Condenser	118.9	108.1	137.1
CRM—Condenser = 0.5 mf	99.2	89.3	118.8
Scaler	98.5	87.1	129.0

All experiments are at the same flow and detention period (10.5 liters per minute, 6.67 minutes). Values are in terms of per cent of 6.67 minutes.

terms of per cent of this constant value. Thus, when the mean is 100% the volumetric efficiency of the tank is 100%. It can be seen that with the Rate-Meter in its normal state the volumetric efficiency is 118.9% against 98.5% for the scaler and 99.2% for the Rate-Meter with the condenser of 0.5 mf, nearly a 20% error. The median value for the uncorrected Rate-Meter curve is also much greater than the other two indicating a large error involved if the time constant is unchanged. The corrected meter values are very close to those from the Scaler indicating that there is very little lag time now present. By many tests similar to this the condenser of 0.5 mf was selected for use and was used in all other experiments of this type. Smaller condensers give larger statistical variations in the curve making interpretation more difficult while still not improving the efficiency noticeably. The Scaler was not used after the calibration was complete for any further work of this kind.

Many dye tests were performed in the manner previously described both with all the baffles in place and in various other combinations. Various rates of flow were also tested. The same tests were repeated with the radioactive tracer. A comparison of the two will now be made. Because with all the baffles in place the lucite tank appeared to be nearly 100% efficient, comparisons on this basis may be more instructive than with any other less efficient combination. A resume of the dye tests of this type is given in Table 4 and for the radiotracer in Table 5. Both groups are over a flow range of about two to one with all the baffles in place. They show many

TABLE 4.—RESULTS OF DYE TESTS ON MODEL SEDIMENTATION TANK

Flow liters/min	Theo. D.P. min.	Vol. Eff. %	Median %	D %	X_{10} %	1DP %	2DP %	FT %
5.47	11.03	86.9	74.1	101.1	4.3	70.3	99.6	3.1
6.72	9.35	84.4	70.5	112.1	3.7	73.5	98.2	2.2
7.54	8.52	84.9	73.2	105.8	3.8	73.5	98.5	2.5
7.73	8.28	91.6	75.7	128.6	3.8	67.3	95.6	2.6
10.37	6.91	90.8	88.8	110.5	4.9	60.7	96.8	1.8
10.53	6.82	93.8	82.1	96.1	4.9	59.8	95.4	3.1
10.70	6.62	95.0	87.6	102.2	5.1	60.1	97.4	2.7
10.93	6.61	89.8	80.8	92.1	4.5	69.5	99.1	1.9
11.01	6.47	93.0	83.1	92.4	4.9	66.2	96.3	2.6
11.10	6.55	91.7	83.2	88.4	4.8	68.3	99.2	2.6
11.15	6.54	92.1	81.1	96.1	4.7	68.6	98.3	2.6
11.16	6.51	94.9	87.1	96.3	4.9	65.5	98.3	3.2
11.19	6.51	99.0	83.2	103.6	4.6	66.1	96.9	2.6
11.21	6.43	93.5	90.3	106.1	5.1	61.8	97.1	2.8
11.26	6.48	93.0	82.5	94.6	4.8	68.0	96.2	2.6
11.26	6.44	96.2	84.5	96.9	4.9	65.2	98.2	2.6
11.29	6.35	94.1	85.5	118.0	4.7	65.4	96.5	2.0
11.36	6.42	92.6	83.0	94.5	4.7	57.2	97.0	2.6
11.48	6.35	96.5	85.1	100.5	4.9	58.3	96.6	3.5
Average (flows greater than 11.000)		94.2	84.4	98.9	4.8	64.6	97.5	2.6

Note:
1. Theo. D.P. is the theoretical detention period or the volume divided by the rate of flow.
2. Vol. Eff. is the volumetric efficiency or the mean of the flow-through curve divided by the theoretical detention period.
3. Median is the point where half of the tracer has passed.
4. D is the decentile range or $X_{90} - X_{10}$.
5. X_{10} is the time for the first 10% of the tracer to pass.
6. 1DP is the per cent of the area that is through in one detention period and 2DP is the per cent of the flow passing through in two detention periods.
7. FT is the time of the first trace of the tracer to be noted.
8. The Median, D, X_{10}, and FT are all expressed as percentages of the theoretical detention period.

TABLE 5.—RESULTS OF RADIOTRACER TESTS ON MODEL SEDIMENTATION TANK

Flow 1/m	Theo. D.P. min.	Vol. Eff. %	Median %	D %	X_{10} %	1DP %	2DP %	FT %	N —
5.98	10.00	83.7	72.5	103.0	4.2	72.5	98.9	2.2	3.7
6.20	9.64	89.2	81.5	101.0	4.6	68.7	98.9	2.3	—
8.78	7.41	98.6	85.5	132.0	4.3	60.7	94.4	2.6	4.7
10.56	6.67	99.2	89.3	118.8	4.8	59.0	94.8	1.8	2.8
10.75	6.54	——	91.2	134.1	4.5	58.0	91.2	0.6	—
11.34	6.50	97.2	86.6	120.9	4.5	61.6	96.6	2.2	—
11.48	6.42	99.3	88.1	135.8	4.7	58.6	93.8	2.3	3.2
11.67	6.34	92.7	78.2	109.9	4.1	65.7	93.0	0.6	—
11.75	6.29	100.1	92.0	118.2	4.7	56.7	93.8	1.2	3.8
11.85	6.24	99.1	84.7	134.9	4.3	61.7	93.7	3.5	—
Average (all flows above 11.00)		97.3	84.4	123.9	4.4	60.9	94.2	2.0	3.5

Note:
1. All notes on Table 4 apply.
2. Tests made with all baffles in place.
3. The 5th, 6th, 8th, and 10th results listed are from data obtained using the scaler. All others are from the use of the Counting-Rate-Meter with a short time constant.

interesting points. However, for the present the accent will be on comparison only. Other significant features will be discussed later. Under these conditions the dye should perform at its best as they are ideal for dye testing. Averages for comparison purposes were made only for the experiments in the flow range of 11 liters per minute as some of the values show change with flow and an average over different flow combinations would not be constructive.

The average volumetric efficiency of the dye tests was 94.2% while that of the radiotracer was 97.3%. The Median (X_{50}) values are identical indicating that the center of area of each curve is the same. The Decentile Range for the dye is 98.9% while that of the radiotracer is 123.9% which is an indication of where the advantage of the radiotracer lies. It shows the greater spread of the cloud that is detectable, that is, the tails of the curves are longer than was found from the dye as the very low concentrations were undetectable with the dyes. This can be further noted in that the first trace was detected earlier by the radiotracers than by the dye (1.96% vs. 2.61%) and also the X_{10} (the ten-percentile) was detected later using the dye (4.36% vs. 4.81%). These results serve to indicate that the early traces are picked up sooner by the radioactive tests. Another indication of the greater sensitivity of the radioactive tracer is that the final tails were detected for longer periods by the radiotracer method. The amount of the tracer passing in one detention period for the dye was 64.6% while for the radiotracer was 60.9%. In two detention periods, for the dye 97.5% of the cloud had passed through but only 94.2% of the radiotracer had passed in the same period. This can be interpreted more clearly perhaps by saying that at the end of two detention periods 2.5% of the dye remained to pass while 5.9% of the radiotracer remained. As the amount of flow that remains in each case must be constant it is evident that the radiotracer shows the longer tails much more efficiently than does the dye. That the tail characteristics are improved by the new tracer is of the utmost importance as will be brought out more clearly later on. With the dye density effects are a factor in the results. This could be clearly seen by visual observation of the passage of the dye through the tank with the aid of the special lighting but there was no way available to measure this effect.

Comparable results are also shown for tests made on the same tank with all the baffles removed. This is an extreme change from

the previously well baffled setup and allows comparison of the performance of the dye and radioactive tracer under two different conditions. The results of these experiments are shown in Table 6.

TABLE 6.—RESULTS OF TESTS ON MODEL SEDIMENTATION TANK RADIOTRACER AND DYE NO BAFFLES

Dye					Radioactive Iodine			
	1.	2.	3.	Aver.	Aver.	1.	2.	3.
Flow 1/m	10.16	10.64	11.41			7.51	8.08	11.15
Theo. D.P. min.	7.04	6.71	6.69			8.23	8.25	6.47
Vol. Eff. %	66.0	60.8	56.4	61.1	73.2	64.0	76.1	72.9
X_{10}—%	3.9	——	4.0	4.0	4.1	4.3	4.3	2.3
Median %	49.4	52.9	41.1	47.8	53.4	51.1	60.8	50.5
D—%	120.2	147.8	158.8	142.3	166.4	174.5	170.1	146.0
One DP %	84.5	77.5	76.8	79.6	71.1	76.5	68.4	70.2
Two DP %	97.4	96.1	98.2	97.2	93.8	97.2	92.7	92.6
First T. % (too early for accurate detection)					0.3	0.4	0.2	0.2
N	1.1	——	1.1	1.1	1.1	1.1	1.1	1.1

Note: For explanation of symbols see Table 6.

These show clearly the same factors indicated previously in the other baffle combination. The volumetric efficiency is higher with the radiotracer (73.2% vs. 61.1%). The medians are still quite close together. The decentile range is higher for the radiotracer (166.4% vs. 142.3%) indicating a longer detectable tail which is also indicated by the fact that larger percentages of the radioactive cloud remain to pass after both one and two detention periods. Additional discussion of these results would be a repetition of those that applied to the well baffled condition but this set of data does further reinforce the previous conclusions made concerning the added benefits obtainable by the use of radiotracers.

These tests made under ideal conditions for the dye especially, serve to indicate the great value of the new tracer in giving a more perfect picture of the actual flow-through curve while at the same time extending the possible fields of use for the tests into work in sewage etc. Because the value of the radiotracer over the dye has been proved, the results of the radioactive iodine tests only will be considered in the following discussion.

N VALUES AND SHORT CIRCUITING

The long sought after measure of settling tank mixing efficiency was first brought out in a paper by H. A. Thomas, Jr. and J. E. McKee entitled Longitudinal Mixing in Aeration Tanks (10). This

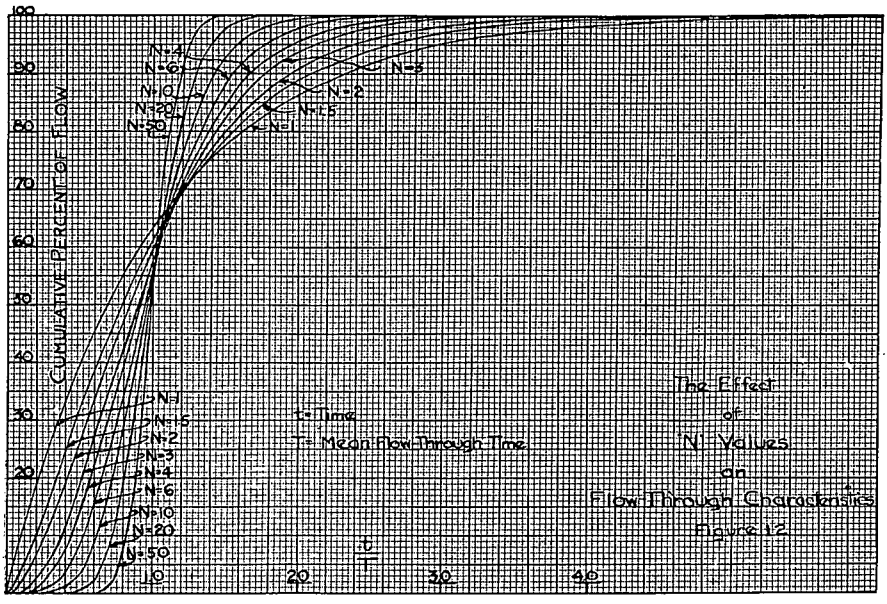


FIG. 12.

measure is denoted by N and can perhaps best be called a short circuiting coefficient. As N approaches infinity the mixing becomes better and better and the smaller the N value the poorer the mixing efficiency and the greater the short-circuiting. The shape of the effluent cloud curve can be expressed as a function of N and appears as follows:

$$y = \frac{1}{T} \cdot \frac{N^N}{(N-1)!} \left(\frac{t}{T}\right)^{-1} e^{-N\frac{t}{T}}$$

where y is the concentration at any time, t, from the initial dosing time and T is the theoretical detention period. This formulation makes the rating of a tank possible and allows comparison with others of various shapes and sizes. The effect of N values on flowing through characteristics can be seen in Figure 12 where values of N are plotted from N of 1 to an N of 50 on a cumulative basis. These curves show the tremendously long tails encountered with poor mixing (low N values) and many other important points. It can be seen that all the curves cross in the vicinity of 63%. At this point all the curves are nearly equal but from there they fan out

widely in both directions. It can also be noted that the change in the curves is greatest per unit of N in the lower ranges of N so that for practical purposes very high values of N are out of the question. For example, the change in characteristics produced in going from an N of 1.0 to 1.5 produces a change about equal to going from an N of 20 to 50, so that changes in the higher ranges are not worthwhile.

N values can be determined in two approximate ways once the shape of the flow-through curve is known. In the first method the ratio of the actual mean to the mode may be set equal to $\frac{N}{N-1}$ and N determined. N can further be defined as the ratio of the peak concentration ($\bar{u}$) to the concentration that would be obtained if the tracer slug were evenly mixed throughout the settling tank (u_o). By taking advantage of this definition, N can also be determined. That is, by knowing the amount of tracer material added and the volume of the tank and the actual mode of the flow-through curve the ratio can be obtained. This ratio can then be set equal to $\frac{N}{(N-1)!}(N-1)^{N-1}e^{-(N-1)}$ and N solved for. As this is rather difficult especially if more than one solution must be made, a better method is to use a plot of the ratio $\frac{\bar{u}}{u_o}$ versus N as shown in Figure 13. Then by knowing the ratio, the corresponding N value can be picked off the chart.

One of the first points that can be noted from Figure 12 was that the curves for low values of N seem to begin in a very short period of time after the dosing time, indicating extreme short circuiting. Further investigation along these lines confirmed this fact. The cumulated equation of the flow-through curve with relation to N value is:

$$P = \frac{N}{(N-1)!}\left[\frac{1}{N}\left(\frac{t}{T}\right)^{N} - \frac{N}{N+1}\left(\frac{t}{T}\right)^{N+1}\right]$$

where P is the per cent of the total flow passed through the basin in time, t, and T is the theoretical detention period.

If P is plotted versus small values of $\frac{t}{T}$ as shown in Figure 14

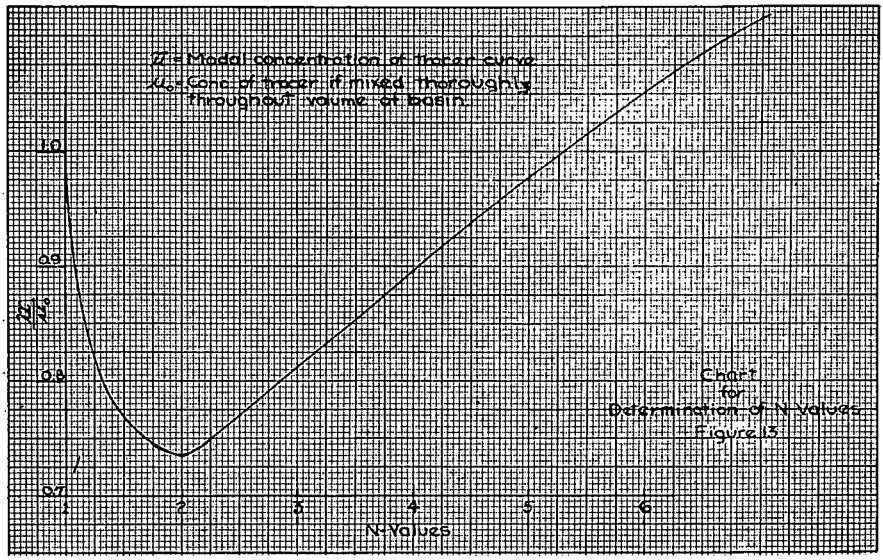


FIG. 13.

the extreme short circuiting with small N values is evident. For example, in 0.01 detention periods 1% of the flow has already passed out if N is equal to 1.0. The values do decrease as N becomes larger but even with an N of 4, which is about the usual value to be expected from a sewage settling tank for example, there is considerable extreme short circuiting.

While this extreme short circuiting is not of major importance in sewage treatment it is of great importance in tanks such as those employed for chlorine contact purposes. These tanks are designed to retain the liquid for a certain period during which the chlorine or other disinfectant has a chance to act and kill the bacteria. If the liquid does not stay in the tank a sufficient length of time this killing will not occur and the effluent will not be safely disinfected.

An investigation into the possible effects of short circuiting in chlorine contact tanks was accordingly made as follows. The time for 100% kill was assumed to be that period after the addition of the disinfectant at which the probability is 50% that no viable organisms remain. If p is the probability of a coliform surviving during any one minute, and 1 — p or q is the probability of dying during any minute and there are n organisms in, say, 1 ml then the expected

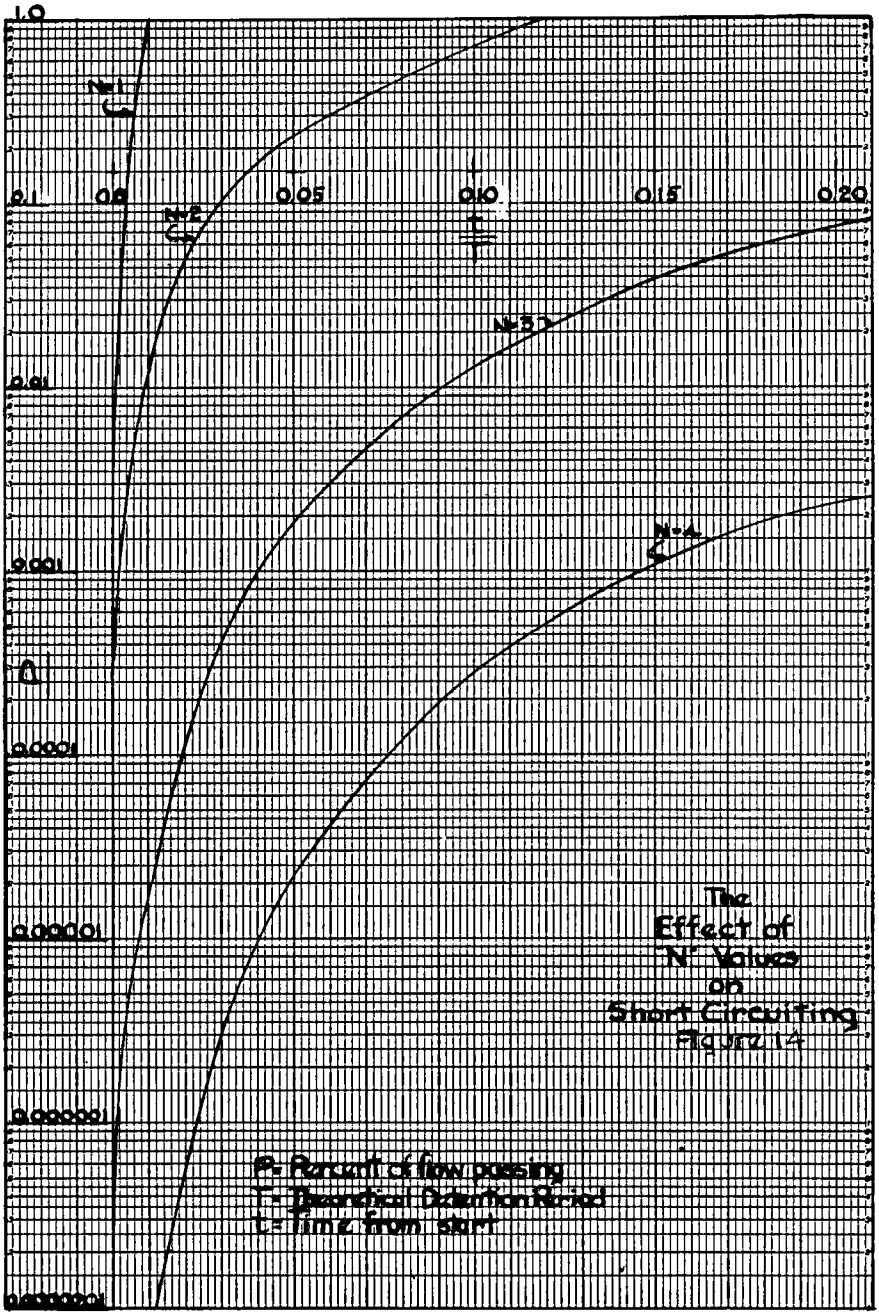

1.0
0.1
0.01
0.001
0.0001
0.00001
0.000001
0.0000001
0.05
0.10
0.15
0.20
N=1
N=2
N=3
N=4
The Effect of 'N' Values on Short Circuiting
Figure 14
P= Percent of flow passing
T= Theoretical Detention Period
t= Time from start

FIG. 14.

number surviving during the first minute is pn. Let $p = e^{-c}$ where c is the 1/p. Then the probability of a coliform surviving t minutes is e^{-ct} or the probability of being dead at the end of t minutes is $(1 - e^{-ct})$. With n coliform organisms the probability that all will be dead at the end of t minutes is $(1 - e^{-ct})^n$. Let the median value of t be t_{50} and it can be shown that t_{50} is equal to $\frac{3.32}{K} \log_{10} 1.44n$ and this is the time for 100% kill as defined above. The K value is the constant for the well known Chick's Law which says that the number dying = (no. alive)$^{-Kt}$.

Examples of bacteria kill by chlorine were obtained from results of work by Wattie and Butterfield (11). In one example the time for 100% kill is given as 10 minutes for E. coli using 0.72 ppm Free Chlorine at a pH of 9.8 with the average initial count being 2,330 coliform per milliliter. From this data Chick's K was determined to be 1.18 for the test in question.

It can be shown that—

$$\text{Final Count} = \frac{\text{Initial Count}}{\left(1 - \frac{KT}{N}\right)^N}.$$

For a detention period of 15 minutes which would normally be considered ample with a 50% safety factor (T) and an N value of 1.0, the final count after passing through the tank is computed by the above formula and using the above data, to be the amazingly high value of 125 per ml. For an N of 2 it is 24 per ml and for an N of 10 it is 0.086 which is still high but residual chlorine in the system would probably soon bring this down to an allowable reading. Similar calculations were made for chloramine treatment with the same type of results. In Figure 15 this is shown graphically for any value of K and T. It is a plot of KT versus the per cent surviving and illustrates the above discussion. It shows that for high N values the decrease in per cent surviving with time is great but for low N values it is much more gradual so that tremendously long detention periods are required to effect a satisfactory kill.

These equations and curves should illustrate the need for proper design and baffling for such tanks as every year there are many cases of typhoid and similar diseases that cannot be traced to any

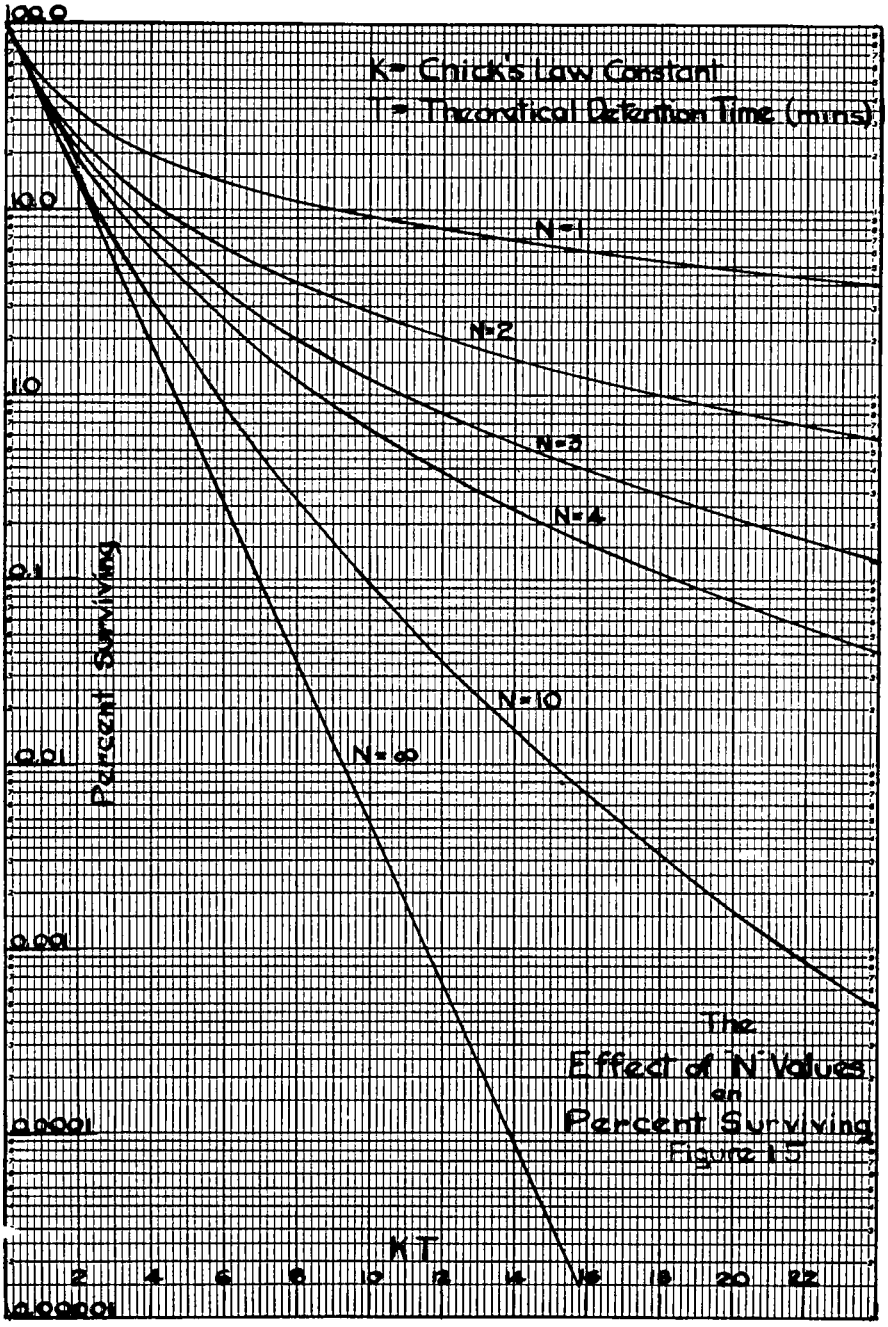

K= Chick's Law Constant
T= Theoretical Detention Time (mins)
100.0
10.0
1.0
0.1
0.01
0.001
0.0001
0.00001
Percent Surviving
N=1
N=2
N=3
N=4
N=10
N=∞
KT
2
4
6
8
10
12
14
16
18
20
22
The Effect of "N" Values on Percent Surviving
Figure 15

FIG. 15.

source and which occur in areas where the water-supply is supposedly properly chlorinated. However, if the contact tank is of poor design some bacteria may get through untreated and cause trouble. Work on proper design of such tanks should be done and this new radioactive tool may be the solution to accurate, easy experiments of this type.

If then, all these hazardous effects are possible, what are the conditions or N values to be encountered in average tank design? As the tank as illustrated previously was thoroughly tested in the laboratory and N values computed some answers can be given using these results. Referring again to Table 5 the N values are shown for the various experiments. The average value for the well baffled tank appears to be about 3.6. As this tank is nearly 100% efficient volumetrically this not exceptionally good N value indicates that this tank is excellent for settling while only mediocre for chlorine treatment. Another opportunity is afforded here to compare the radiotracer with the dye. The average N computed from the dye experiments is 2.6. (Results not shown.) This is lower than that from the radioactive tracer because the tail was shortened up by the non-detection of part of the tail. This causes a smaller value of the mean and a corresponding decrease in N because the mode may be considered to be unchanged. Thus higher N values can be expected from radiotracers than from dye tests.

To test the effect of baffles on N values various combinations were used in the model lucite tank. All tests were made at the same flow rate so as to be comparable. The various arrangements of baffles are as shown in the small sketches and the results are illustrated in Figure 16. Examples of the curves for the extreme conditions are shown in Figure 17. From these curves a great difference in flow characteristics is evident especially in the large amounts of the flow passing through the tank so quickly with the poorer baffle arrangements. The results are also shown in Table 7. The N value for each setup is shown and they range from 4.5 to 1.0 which serves as a good illustration of the baffles' value in mixing. It can be seen that with a setup similar to this any tank could be designed for proper baffling by a few laboratory tests. From the results it is seen that the downstream baffle has very little effect on any of the measures and that the upstream one is of the greatest importance to good mixing and tank efficiency as all these values fall off considerably when it is

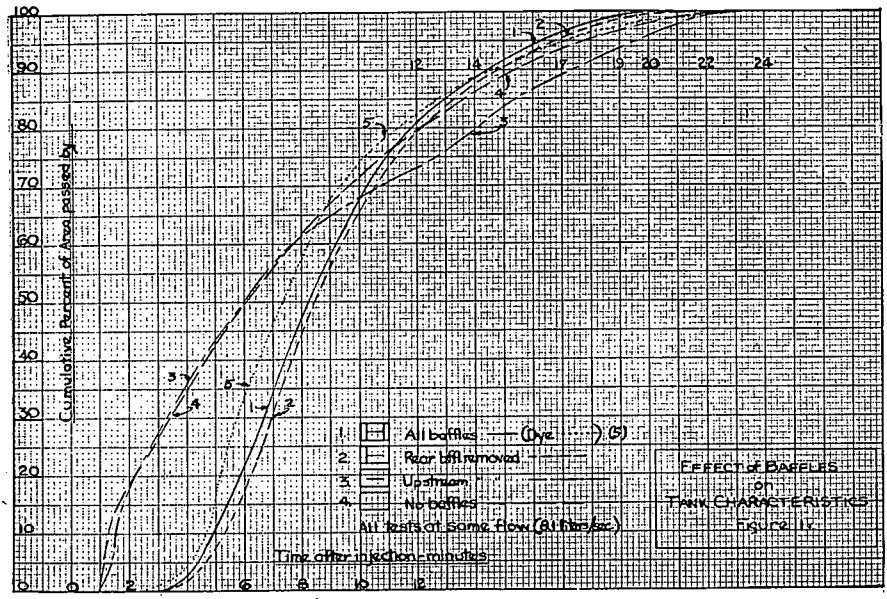

Cumulative Percent of Area passed by
Time after injection-minutes
1. All baffles — (Dye) (5)
2. Rear bff removed
3. Upstream
4. No baffles
All tests at same flow (Al bar/sec)
EFFECT OF BAFFLES ON TANK CHARACTERISTICS
FIGURE IV

FIG. 16.

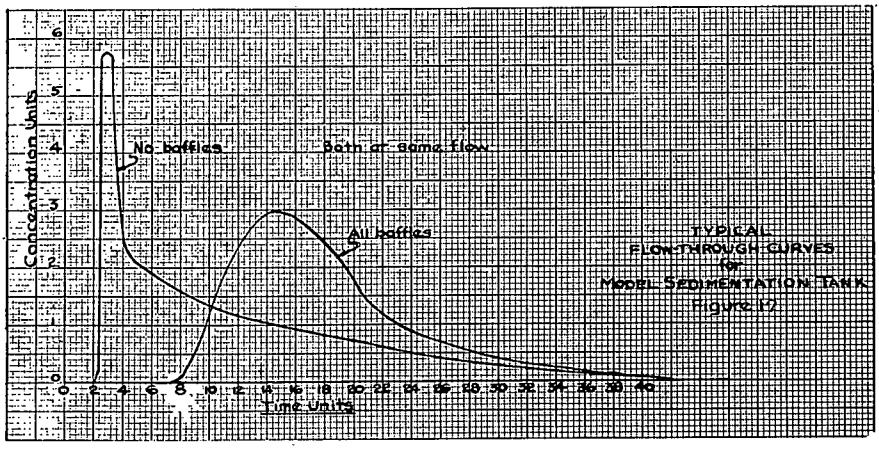

Concentration Units
No baffles
Both at same flow
All baffles
Time Units
TYPICAL FLOW-THROUGH CURVES FOR MODEL SEDIMENTATION TANK
Figure 17

FIG. 17.

TABLE 7.—EFFECT OF BAFFLES ON BASIN PARAMETERS

	I All baffles	II Downstream removed	III Upstream removed	IV No baffles
Vol. Eff.—%	98.2	100.2	83.4	76.1
Decentile Range—%	117.8	118.1	195.5	170.1
Median (X_{50})—%	88.6	91.5	60.5	60.8
X_{10}—%	48.1	51.7	5.8	4.3
One DP—%	58.8	57.8	65.6	68.4
Two DP—%	97.0	96.0	87.7	92.7
First Trace—%	2.5	2.7	0.2	0.2
N Value	4.5	4.7	1.1	1.1

Notes:
1. All tests made with radioactive tracers.
2. See Figure 16 for baffles arrangements.
3. For explanation of symbols see Table 4.
4. Rate of flow and detention period constant for all tests.

removed. The center baffle also exerts little effect. The variance of the various factors are evident from the table and do not need to be discussed. It may, however, be noted that the per cent passing through in one detention period increases with poorer baffling while the per cent passing in two detention periods increases, indicating the enlargement of the tail of the curves as the baffling and mixing become poorer. This is further indicated by the increase in the Decentile Range which increases quite rapidly as N becomes smaller. A glance back at Figure 12 would show that this lengthening of the curve was predicted from the theoretical treatment and this serves to indicate the value of the previous analysis.

This series of experiments also serves to show by experimental means the effect of N values on extreme short circuiting. For the setup where N was 4.5 the first trace of the cloud was noted at 2.5% of the theoretical detention period while when N was 1.1 the first trace appeared in 0.182% of the detention period. If this had been a contact tank with a detention period of fifteen minutes some of the flow would be getting only 1.6 seconds of treatment. Short circuiting results from the series of experiments with no baffles at all which were described previously, give an average value very close to this, while the average for the group with complete baffling was 2.1%. These indicate, what has been shown previously about the value of good design to prevent this dangerous condition. This will be further brought out in the discussion of the field tests.

As it has been shown that the dye experiments do not detect the tail of the curve as accurately as do the radiotracers some computa-

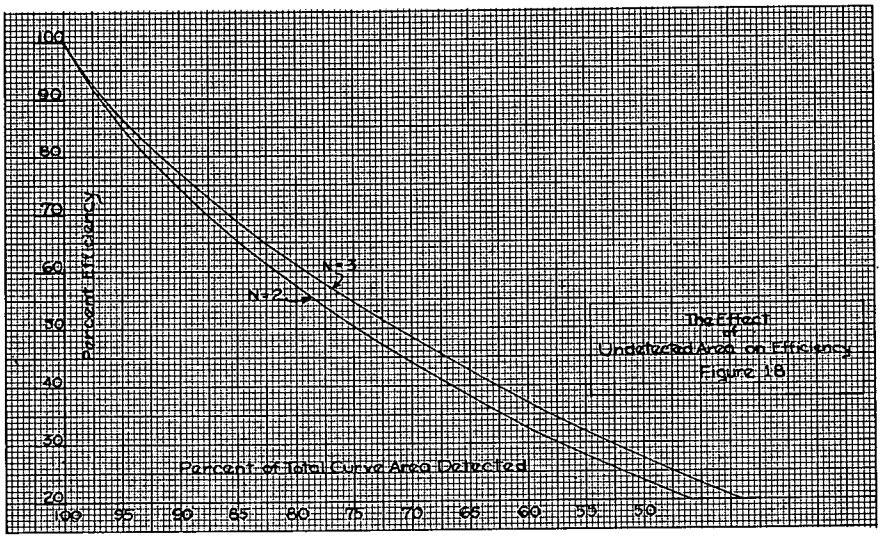


FIG. 18.

tions have been made showing the effect of removal of various percentages of the true flow-through curve which is what, in effect, is done by the non-detection of low concentrations of the dye or salt. The results are shown in Figure 18 for curves of N values of 2 and 3. The effect of removal of various percentages is plotted against the change in volumetric efficiency that it would bring about. These figures indicate that with as little as 5% of the total area of the curve removed or undetected the efficiency will be lowered to about 85% and even greater inefficiencies occur as the per cent undetected is increased. This fact is brought out merely to illustrate the point that the non-detection of part of the total curve can play a very important role in giving inaccurate results to such experiments as these. It is therefore to be desired that the low concentrations be carefully detected as is done with the radioactive tracers.

EFFECTS OF MIXING

Baffling is in a sense, mixing, so it was reasoned that the effects of baffles might be duplicated by the use of mixing and thus the effect could perhaps be studied to greater advantage.

To do this a long lucite tube approximately 2½ inches in diameter was fitted with a longitudinal mixing rod. This rod ran the entire length of the tube of about ten feet. It was rotated by gearing

at one end and the speed of rotation was variable. An overflow weir at the effluent end controlled the flow. The flow was measured volumetrically and the radiotracer was injected under slight head as before. The Geiger tube was placed near the affluent and operating from the counting-rate-meter and recorder where the curve of the passing tracer concentration was plotted. Three different flow rates were tested, each one with five changes in the amount of mixing. Each step in mixing speed was a constant increase. The interesting results are as follows. They are shown in Table 8.

The effect of mixing on the volumetric efficiency was marked in each case. The efficiency decreases with the increase in mixing. The N value, as would be expected, increases in each case with the increase

TABLE 8.—EFFECT OF MIXING ON CURVE PARAMETERS

	Vol. Eff	Dec. Range	Median	X_{10}	N
	A. Flow 1.40 liters per minute				
No Mixing	84.5	121.9	65.9	39.2	2.38
Very Slow	80.3	114.8	67.3	39.2	2.37
Med. Slow	——	96.7	61.8	37.8	3.03
Med. Fast	77.8	90.8	72.8	43.8	4.38
Very Fast	75.7	57.1	67.4	44.3	4.48
	B. Flow 0.75 liters per minute				
No Mixing	65.6	57.7	60.6	42.9	2.14
Very Slow	63.6	67.2	54.8	37.5	4.01
Med. Slow	——	49.3	46.2	36.4	5.52
Med. Fast	58.0	51.1	50.0	39.1	5.65
Very Fast	59.6	38.0	53.6	44.0	6.59
	C. Flow 0.38 liters per minute				
No Mixing	79.0	79.3	72.6	45.7	2.82
Very Slow	75.8	——	72.6	49.1	——
Med. Slow	62.2	61.6	——	38.5	3.18
Med. Fast	60.8	——	55.8	17.4	3.93
Very Fast	57.2	28.3	54.0	45.3	7.07

All values except N are given as per cent of the theoretical detention period.

in mixing. The median value is practically constant for the highest flow rate at the various mixing speeds while at the lower flows it appears to decrease with increased mixing. No definite variation can be noted. The ten-percentile also remains practically constant at all flows and mixing rates. The decentile range appears to decrease with increased mixing which in line with all the previous discussion of both theoretical and experimental results showed that the spread of the cloud curve is decreased as the mixing increases. The increased mixing then tends to bring the curve closer and closer together around the Median which as shown here remain somewhat constant.

The effect of mixing is as anticipated on the value of N showing that N is a true measure of mixing as it was defined originally and this test serves to indicate that the original definition was correct. Just why the effect on the volumetric efficiency should proceed in the inverse direction is not clear but it definitely does appear to do so. Further work is needed to clear this up. The variation of these parameters with flow does not appear to be marked as not sufficient flow rates were used to definitely establish any trends. Most of the parameters, however, show the same tendencies at all three flow rates. It is of interest that the change from no mixing to very slow mixing (35 rpm) produced very little change in any of the parameters indicating perhaps that until sufficient energy of mixing is produced there is little effect upon the flow pattern.

VARIATION OF PARAMETERS WITH FLOW

Examination of the data in Tables 4 and 5 for both the dye experiments and those with radiotracers seem to indicate that there is some variation of the various parameters with changes in flow. This is shown graphically for some of them in Figure 19. Because the tests were not performed specifically to test this variation, the

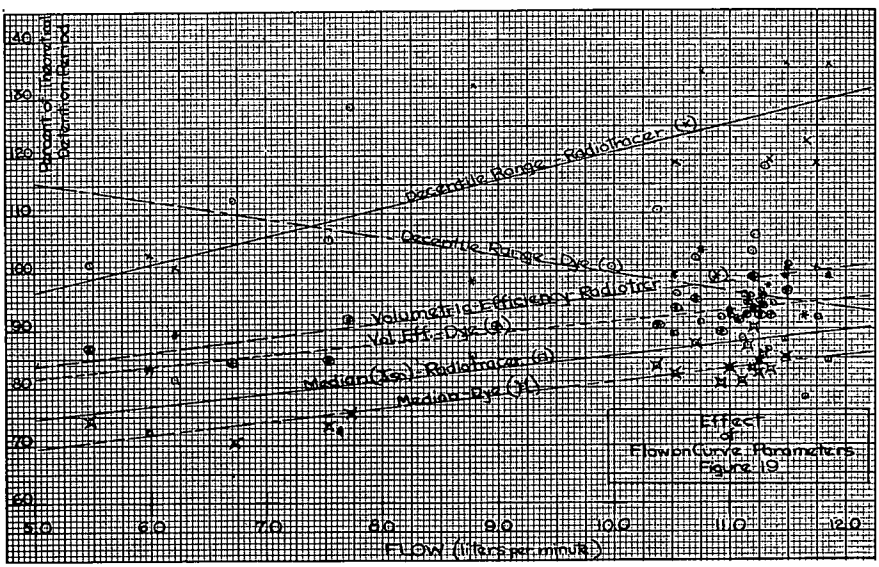


FIG. 19.

distribution of points is not too even, but despite this there does seem to be a trend toward an increase in the volumetric efficiency as the flow changes with both the dye and the radioactive tracer. All tests shown are from tests made with all baffles in place. Incidentally this graph shows very clearly the fact that the radioactive tracer gives a higher value for the efficiency (about 5%) than that from the dye due, of course, to the greater length of the tail detected. This increase in efficiency with flow is on the safe side for tank designers as the lower flows give longer detention periods and while percentage-wise the tanks are less efficient in terms of time the values may be nearly the same. It can also be seen that the median value rises at a rate nearly parallel to that of the efficiency with increasing flow and the difference between the dye and the radiotracer is still evident. One interesting factor is that while the decentile range increases with flow for the radio-iodine it decreases with flow in the dye tests. This can perhaps be explained by the fact that at low flows there is less relative mixing and the tails are really longer and stronger relatively than at high flows and thus are detected for longer periods by the dye at low flows than at higher ones. Further investigation of this point may reveal a more feasible explanation.

Whether these changes of efficiency etc. are important in actual field work is not known but there is an indication that with changes in flow the tank may behave differently. Further tests are needed to confirm these indications.

FIELD TESTS

The work in the field was undertaken mainly to show that the technique so well demonstrated in the laboratory would apply equally as well in the field where dilutions and flows were so much greater.

The first basin-type test was performed at an activated sludge plant and the test was made on the primary sedimentation tank. This was a rectangular tank approximately 60 feet x 35 feet with an average depth of 8.75 feet. At the effluent end sewage passed over a weir and thence into a rectangular channel. A tracer dose of about 5.4 millicuries was introduced at the effluent end of the grit channel immediately preceding the settling tank. Samples were taken in the effluent channel immediately after the weir where flow from all parts of the tank was thoroughly mixed. The results of this test were extremely interesting. The flow-through curve is shown in Figure 20 as well as the cumulated curve in Figure 21.

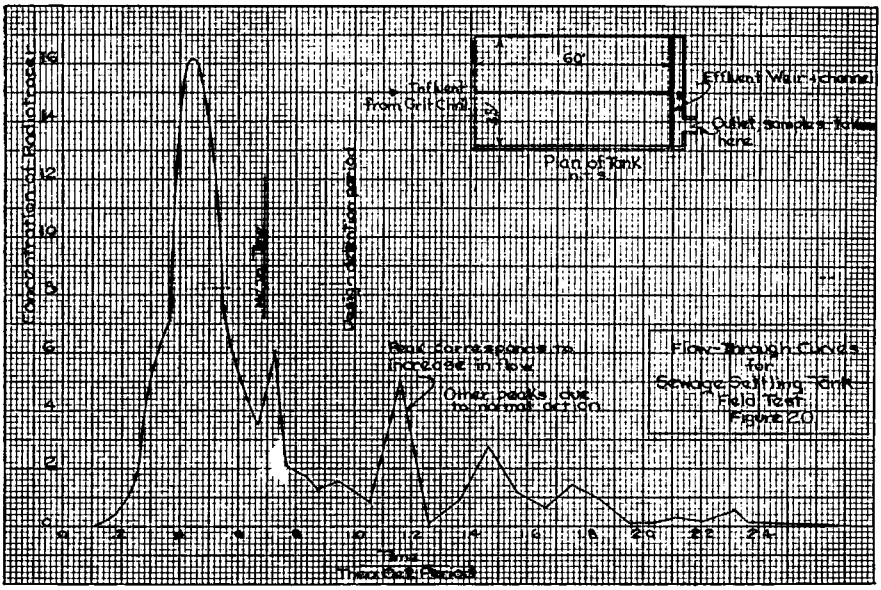

Flow-Through Curves for Sewage Settling Tank Field Test Figure 20
Plan of Tank
Effluent Weir & channel
Effluent from Grit Chamber
Peak corresponds to increase in flow
Other peaks due to normal action
Time
Theoretical Detention Period

FIG. 20.

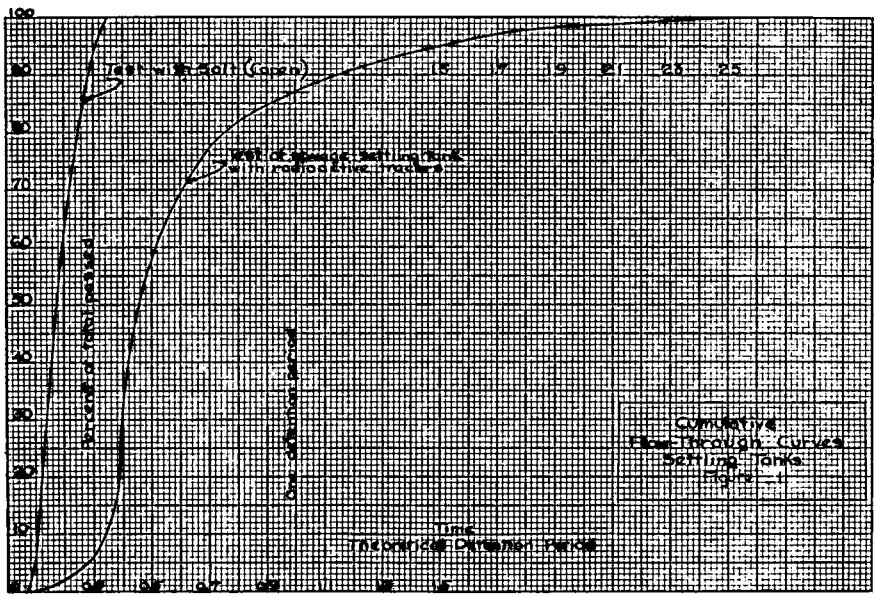

Test with Salt (Cocoa)
Test of sewage settling tank with radioactive tracers
Percent of total passed
Cumulative Flow-Through Curves Settling Tanks Figure 21
Time
Theoretical Detention Period

FIG. 21.

With the average flow through the tank being 2.8 MGD the theoretical detention period was 63.8 minutes, allowing for the pipe volume from the grit channel to the tank. From the sample results the mean time was 41.0 minutes giving a volumetric efficiency of 64.3%. The time for the first 10% to pass through was 22 minutes and for 90%, 72 minutes or a decentile range of 50 minutes. Half the tracer passed through the tank in 28.1 minutes and in one detention period 86.1% had passed through. These values can be easily seen from the cumulative plot. The N value for this tank was computed to be 3.0. Short circuiting was very rapid as the first sample at ten minutes after dosing was radioactive and if earlier ones had been taken an earlier appearance would probably have been noted. Thus these results indicate that this tank which meets nearly all criteria for good design is not efficient (64.3%). Fifty per cent of the flow gets only 28 minutes of treatment while only 14% gets the full 60-minute design detention or better. An N of 3.0 indicates fair mixing. It is thus seen that this tank which is well designed does not perform as it should and this indicates that there is a definite discrepancy in present design methods.

It is of interest to compare the results of this test with those made by Capen (6) on other settling tanks in the field using salt. As was mentioned previously the average efficiency he found was 22.8% and ranged from 7% to 48% in ten tests described. The tanks appear to be of ordinary design and these low efficiencies appear to be much too small to represent the actual conditions. The possible reason for this can be seen from the data on one test that is shown here. The cumulative curve is shown in Figure 21 and can be compared with the curve from the tank test previously described made with radioactive tracers. Note the extremely short time length of the curve in relation to the detention period and also compared to that of the other settling tank curve. The detention period should have been 4 hours and 34 minutes and from the data was found to be only 50 minutes giving an efficiency of only 19%. It is fairly evident that this is an extremely good example of what occurs when salt or dye is used. Only the top of the curve is detected resulting in the low efficiencies which result when part of the curve is removed as previously explained. This should be further proof of the advantages of the radiotracer in this type of work.

Another similar test was performed on a circular clarifier that

was operating as a secondary sedimentation tank after trickling filters. Here, the dosing was not done as efficiently as could be desired due to the physical setup but the slug was distributed as evenly as possible around the inside of the center distribution well where the sewage enters the tank. Samples were taken at the effluent channel. About the same dose was used as before. The tank was 100 feet in diameter with a side-water depth of 10 feet. The flow during the test averaged 5.0 MGD giving a detention period of 181 minutes.

From the test results the mean time was 25.3 minutes or a volumetric efficiency of only 14%. The first trace was noted in three minutes and ten per cent had passed through in 12 minutes. Fifty per cent passed by in 19.5 minutes and by the end of 57.5 minutes 90% was through. The N value was 3.5 indicating fairly good mixing. This test cannot, however, be considered as too conclusive due to the perhaps poor dosing methods and some error in the flow-rate may be possible. However, it does seem to indicate very poor efficiency for this type of tank and it illustrates very clearly the extreme short circuiting encountered, with the first trace noticeable in 1.66% of the detention period, very similar to that found in the model tests. It further shows the practicality of the tests made with radioactive materials as tracers.

Before going further it may be well to note that all these tests in the field were made with the full knowledge and permission of all authorities including the Atomic Energy Commission which supervises the use of such material very carefully to prevent any dangerous conditions from existing. All safety precautions were taken and no solutions above allowable concentrations were used in any of the experiments. Most of the tests were actually carried out at a lower level of activity than is found in many springs whose waters are used as health tonics.

Another basin test was performed in a pond to determine its flow-through characteristics and to determine whether such a test was actually possible in such a large body of water. The pond selected for use is as shown in Figure 22 with a volume of 5.93 MG and an area of 4.9 acres. It was not ideally shaped for such a preliminary test and was not efficient because of the large backwater area, but due to the weather conditions it was the only one suitable and available. Dosage was made with about 20 millicuries at the influent end, where shown, and samples were collected at the outlet.

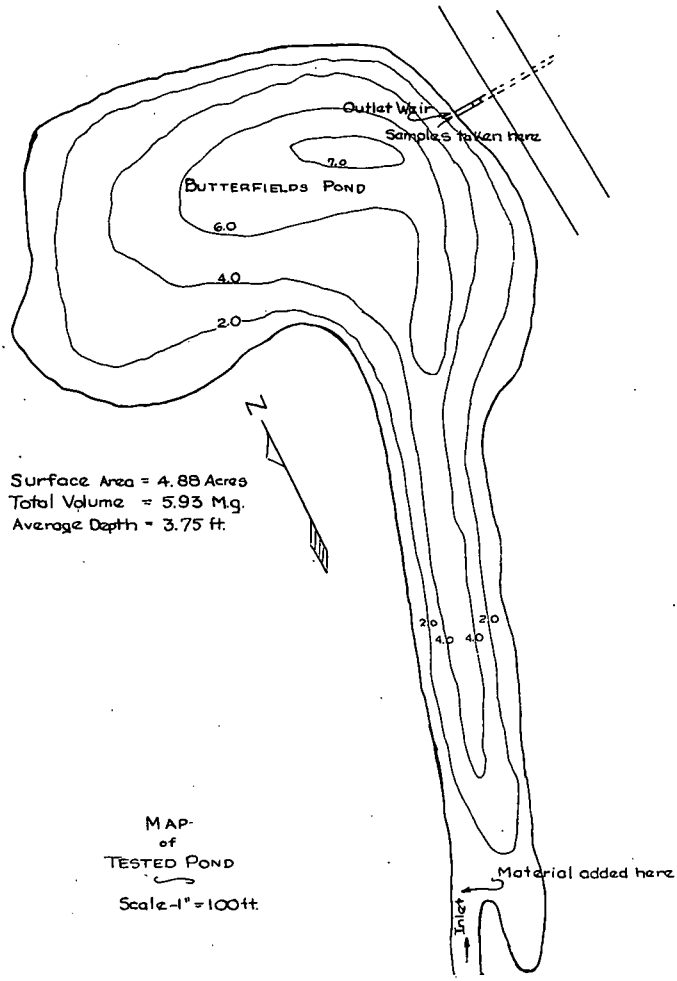


FIG. 22.

The average flow over the testing period was 1.27 cfs. giving a theoretical detention period of 7.2 days. The actual detention period was 3.0 days so the efficiency was 42.4%. The N value was 2.6 showing rather poor mixing. Short circuiting was also present as 10% of the cloud passed through in 1.2% of the detention period. Fifty per cent passed in 2.0 days and the decentile range was 6.0 days. The actual results have no significant interest except that they

do show that such a test is possible and can be easily and accurately performed. This is the first test of this kind ever made on a pond to determine its flow-through characteristics, as far as is known. The performance of ponds has always been a mystery to engineers as performances vary from pond to pond and from time to time. This test can then serve to uncover some of these factors in a convenient manner. If salt were used at least a half-ton would be required and density effects would be tremendous with that quantity in such a small flow. Only 5 ml of radioactive tracer was used in this test. The future for such pond testing should reveal many interesting results about pond behavior that have hitherto been unknown.

At the Pondville State Hospital in Norfolk, Mass., tests were made to trace a slug of radioactivity through the sewage treatment plant. A dose was flushed from the last toilet on the line from the hospital to the treatment plant and samples were taken as it reached the plant. It then passed on into an intermittent dosing settling tank which discharged onto sand filters. The slug was traced to the sand beds but none was found in the effluent from these beds. The reason for this is unknown and no explanation will be offered except for possible experimental error which is always possible at such low concentrations. Further tests should be made to clear this up. These tests at Pondville had no theoretical interest and no results are included. They did show that long detention had no effect on the removal of the radioactivity which had been predicted in the laboratory and they further served to show that this would be a useful tool for tracing flow through a plant or some other similar setup.

In a test on a trickling filter installation, which was not strictly a basin test, the time of contact of the sewage with the media was determined. A dose of radioactivity was dumped onto the media in conjunction with the passing of the distributor arm at one point and samples were taken in the effluent. These showed that for this low rate filter that the mean time of contact including travel through the underdrains was 7.65 minutes with 50% passing in 5.86 minutes, 10% in 4.33 minutes and 90% in 10.0 minutes. The first trace was noted in one minute. While these results do not have any special significance they do indicate that this new tool can be used in a variety of ways that may help to improve the design of many types of treatment by uncovering many hitherto unknown factors.

Thus these field experiments have shown that the same accurate

results can be obtained in field work as was found in the laboratory. A large variety of field tests were performed and many others are possible. All the ones tested seem to indicate the usability and value of the radioactive tracer.

Conclusions

It is believed that the radioactive tracer is a tool long needed by hydraulic and sanitary engineers in making the type of tests herein described. Its powers are unaffected by changing chemical and physical conditions and can be detected in very low concentrations, much lower than is possible with dye or salt. This tracer gives a more accurate picture of the actual flow-through curves of basins and conduits by bringing out the longer tails of the curves which go undetected by other means. Density effects are improbable as only very small amounts of the material is required. In many cases, especially in model studies, the radiotracer may be followed through the system without disturbing the flow pattern by taking readings through the pipe or tank walls. These, then, are the main advantages that were stressed throughout the paper in various forms.

With all these advantages and probably more to be uncovered in the future the possibilities for this new tool of the engineer seem very bright indeed. Adaptations of this method can undoubtedly be made to many problems of a similar nature and there are still many leads given here that have not been thoroughly investigated. It is hoped that the work begun here will be continued so that more light may be shed on unknowns in the hydraulic and sanitary fields.

Acknowledgments

This work was made possible through the John R. Freeman Fund of the Boston Society of Civil Engineers. It was a valuable opportunity for being given the chance to carry out this instructive work. I wish to especially thank the committee, and H. M. Turner, chairman, for their kind help and cooperation. I wish also to express my great appreciation for the help given me by Prof. H. A. Thomas, Jr., who aided and guided the work, and Prof. E. W. Moore and others of the staff of the Harvard Graduate School of Engineering, Sanitary Engineering Department. The Sanitary Engineering Division of the Massachusetts Department of Public Health through its chief engineer, Mr. Arthur Weston, lent valuable assistance in the

field work and provided funds for some of the most important testing. Their help was greatly appreciated and added much to the paper. Much editing, typing and other invaluable help was given by my wife.

Thanks and appreciation are owed to many others who made parts of this paper possible. It is hoped that they will consider themselves included in my appreciation.

It is hoped that this paper will be of some future value and will lead others to continue this work in the development of the radioactive tracer in flow tests.

List of Symbols

SYMBOL OR ABBREVIATION	MEANING OR UNITS
cm.	centimeter
rem.	roentgen-equivalent-man
rep.	roentgen-equivalent-physical
r.	roentgen
Mev	Million electron voltes
mc	millicurie
RC	Resistance-Capacitance
mf	micro-farad
cpm	counts per minute
mm	millimeter
ml	milliliter
mg	milligram
gps	gallons per second
R	Reynolds Number
cfs	cubic feet per second
MGD	Million Gallons per Day
ppm	parts per million
cc	cubic centimeter
l	liters
ln	natural log
DP	Detention Period
CRM	Counting-Rate-Meter

The above are arranged in the order that they appear in the text.

List of References

1. C. M. Allen, & E. A. Taylor, "The Salt-Velocity Method of Water Measurement", Transactions of the American Society of Mechanical Engineers, vol. 45, 1923, p. 285.
2. L. J. Hooper, "Salt-Velocity Measurements at Low Velocities in Pipes", Transactions of the American Society of Mechanical Engineers, vol. 62, 1940, p. 651.
3. Martin A. Mason, "Contribution to a Study of the Allen Salt-Velocity Method of Water Measurement", Journal of the Boston Society of Civil Engineers, vol. 27, 1940, p. 207.
4. B. J. Groat, "Chemi-Hydrometry and Its Applications to the Precise Testing of Hydro-Electric Generators", Transactions of the American Society of Civil Engineers, vol. 80, 1916, p. 951.

5. A. Barbagelata, "Chemical-Electric Measurement of Water", Proceedings of the American Society of Civil Engineers, vol. 54, 1928, p. 789.
6. C. H. Capen, Jr., "Study of Sewage Settling Tank Design", Engineering News Record, vol. 99, p. 833.
7. V. F. Hess, "On the Use of a Radioactive Tracer Method in Measurement of Water", Transaction of the American Geophysical Union, Part II (Hydrology) April 23, 1943 (Proceedings of the Royal Dublin Society, Vol. 16, 1922).
8. S. Karrer, D. B. Cowie, & P. L. Betz, "Use of Radioactive Tracer in Measuring Condenser Water Flow", Power Plant Engineering, December, 1946, vol. 50, no. 12, p. 118.
9. C. F. Johnson, "Determination of Kutter's N for Sewers partly Filled", Transactions of the American Society of Civil Engineers, vol. 109, 1944, p. 223.
10. H. A. Thomas, Jr., & J. E. McKee, "Longitudinal Mixing in Aeration Tanks", Sewage Works Journal, January, 1944.
11. Wattie, Butterfield, "Public Health Reports", December 29, 1944.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

OCTOBER 19, 1949.—A regular meeting of the Boston Society of Civil Engineers was held this evening at Social Hall, Tremont Temple, Boston, Mass., and was called to order by President, Harrison P. Eddy, Jr., at 7:00 P.M. This was a joint meeting with the Transportation Section, B.S.C.E.

President Eddy announced that the minutes of the September meeting would be published in the forthcoming issue of the JOURNAL, after which there will be opportunity for their correction. The reading of the minutes to be waived unless objection is raised.

President Eddy announced the death of the following member:

Bertram D. Brewer, who was elected a member May 20, 1896, and who died August 8, 1949.

President Eddy stated he had been requested by Mr. Miles N. Clair, chairman, New England District Council, American Society for Testing Materials, to announce that a meeting on "'Solar Heating" would be held on October 24, at which all members would be welcome.

President Eddy announced this was a joint meeting with the Transportation Section and turned the meeting over to William L. Hyland, Chairman of the Transportation Section, to conduct necessary business matters of that Section.

At the conclusion of the business matters of the transportation Section Mr. Hyland introduced the speaker of the evening.

Speaker—Mr. John D. M. Luttman-Johnson, Engineer, Fay, Spofford and Thorndike.

Subject—"Harbor Engineering, with Particular Reference to Development of Port Elizabeth, Union of South Africa".

The talk was illustrated with slides.

Fifty-two members and guests attended this meeting.

The meeting adjourned at 9:00 P.M.

ROBERT W. MOIR, *Secretary*

NOVEMBER 9, 1949.—A joint meeting of the Boston Society of Civil Engineers and of the Northeastern Section of the American Society of Civil Engineers was held this date at Northeastern University, Boston, Mass. Members of student chapter and civil engineering students of the New England Colleges were especially urged to attend.

A catered dinner was held in University Commons Hall, Northeastern University from 6:00 to 7:00 P.M. Student delegations were present from, Northeastern University, Harvard University, Massachusetts Institute of Technology, Tufts College, Rhode Island State College, University of Connecticut and Dartmouth College.

In the absence of President Harrison P. Eddy, Jr., and first Vice Presi-

dent Thomas R. Camp, Vice President John B. Wilbur presided and called the meeting to order at 7:30 P.M.

Vice President Wilbur extended a cordial welcome to the students and expressed appreciation of the cooperation of the officers of the student organizations and of the faculty members in making this event so successful.

Secretary, Robert W. Moir, presented the report of the Tellers, E. F. Childs and H. L. Kinsel, on the ballot on "Recommended Procedure for Securing Professional Engineering Services by Public Authorities", action on which was taken at the Society meeting, September 28, 1949, referring this matter to the whole membership of Society. He stated that there were 276 ballots cast in favor of and 8 opposed to the endorsement. Vice President Wilbur declared that the endorsement has therefore been adopted as follows:

Recommended Procedure for Securing Professional Engineering Services by Public Authorities

Engineering services are personal services and are professional in their nature, like the services of doctors and lawyers. Engineering services should be employed on the basis of competence, integrity, and mutual confidence between the public authority and the engineer. These conditions may not be fulfilled if the engineering service is selected on a price basis. Competitive price bids lose their significance unless based on precise specifications; and these cannot be written for the intangible qualities of training, experience, ability, judgment and integrity required of the engineering profession.

Consequently, it is contrary to the best public interest to procure professional engineering services on the basis of competitive prices, as practiced by a few municipalities; because this implies an obligation to employ the low bidder regardless of whether he is best qualified.

One of the principal reasons for obtaining competitive prices is the belief that the law so requires. However, the courts have ruled that competitive bidding for professional services is not required. (See McQuillan on Municipal Corporations, Vol. 2, sec. ed. Revised, p. 1182, sec. 1292). Furthermore, the results of a survey completed last year by the Society of Civil Engineers, by means of questionnaires sent to the legal counsel of all Massachusetts Municipalities having a population of 5,000 or more, showed that over 90 per cent of those who replied did not interpret their municipal ordinances to require competitive bidding for professional engineering services.

Most public works projects must start with a preliminary engineering investigation and report upon the need, the functional design and extent of the project, the cost of construction and operation, methods of financing and assessment of costs. Since the preliminary investigation may set the pattern for the ultimate success or failure of a project, it is deserving of engineering of the highest order. It is of utmost importance to the public that the cost of the preliminary studies should not be a condition in selecting the engineer for a project.

The fee for professional engineering services is but a small part of the cost of a project and the difference in the fees charged by a competent and by an incompetent engineer can be saved many times over in the entire project cost, if the project is well conceived and well executed.

It is to the best interest of the public that a Public Authority which does not already have satisfactory engineering associations engage professional engineering services by the following procedure:

1. Determine by inquiry from properly informed sources the names of one or more engineers or engineering firms, who may

be qualified and available for the particular engagement.

2. Interview one or more of said engineers and examine his or their qualifications.
3. Select one engineer and negotiate with him an agreement as to the nature and extent of the services to be rendered and the basis of payment. If a question arises as to the reasonableness of the fees to be charged the Public Authority should seek advice from recognized authorities such as the American Society of Civil Engineers. If an initial agreement for engineering services upon a project is limited to a preliminary investigation and report, then the agreement should commit the engineer as to availability and as to limiting fees, in case complete professional engineering services are desired at a later date.

Vice President Wilbur introduced Arthur L. Shaw, President of Northeastern University Section, A.S.C.E., and asked him to conduct any matters of business required by that Society.

Vice President Wilbur introduced the speaker of the evening, Rev. Daniel J. Linehan, S.J., Seismologist-in-Charge, Seismological Observatory, Weston College, who gave a most interesting illustrated talk on "Earthquakes and the Application of Seismic Surveys." A question period following the talk proved of interest also.

A rising vote of thanks was given the speaker.

Four hundred members and guests attended the dinner and four hundred forty-five attended the meeting.

The meeting adjourned at 9:20 P.M.

ROBERT W. MOIR, *Secretary*

DECEMBER 14, 1949.—A Joint Meeting of the Boston Society of Civil Engineers and the Structural Section, B.S.C.E., was held this evening at Chipman Hall, Tremont Temple, Boston, Mass., and was called to order by President Harrison P. Eddy, Jr., at 7:00 P.M.

President Eddy announced that the minutes of the October and November meetings will be published in the forthcoming issue of the JOURNAL, after which there will be opportunity for their correction. The reading of the minutes will therefore be waived unless objection is raised.

President Eddy announced the death of the following member:

Luther H. Bateman, who was elected a member February 21, 1894, and who died October 26, 1949.

The Secretary announced the following had been elected to membership:

Grade of Member.—Percy L. Coleman, Frank E. Fahlquist, Stanislaw J. V. Gawlinski, Thomas J. Rouner.

Grade of Junior.—Harold J. Publicover.

Grade of Student.—William E. Ingram, Reidar J. Formo.

President Eddy announced that this was a joint meeting with the Structural Section and turned the meeting over to Ernest L. Spencer, Chairman of that Section to conduct necessary business matters for that Section.

At the conclusion of the business matters of the Structural Section Mr. Spencer introduced the speaker of the evening.

Mr. J. J. Jenkins, Jr., Chief Engineer, J. R. Greiner Company, Consulting Engineers, Baltimore, Maryland.

Subject—"Design Problems Attendant to the Mystic River Bridge".

The talk was illustrated with slides.

One hundred and six members and guests attended the meeting.

The meeting adjourned at 8:00 P.M.

ROBERT W. MOIR, *Secretary*

SANITARY SECTION

DECEMBER 7, 1949.—A regular meeting of the Sanitary Section was held in the Society Rooms and called to order by Chairman Knowlton at 7:15 P.M. Thirty-two members and guests attended a dinner at Patton's Restaurant preceding the meeting. Present at the meeting were 64 members and guests.

Minutes of the October 5th Meeting were read and approved.

Chairman Knowlton announced future meetings:

A Joint Meeting with the Main Society to be held January 25th at which Drs. Redfield and Ketchum would discuss the subject of Ocean Currents.

Next regular meeting to be March 1, 1950, at which there is to be a discussion of the Stream Pollution Program of Massachusetts and New England.

Mr. F. M. Cahaly moved that a Nominating Committee be elected comprising:

Messrs. Murray H. Mellish
Walter E. Merrill
George F. Brousseau

Mr. Stephen Hazeltine moved that the names of—

Messrs. George G. Bogren
Lincoln W. Ryder
A. A. Thomas

be also considered.

Moved by Thomas A. Berrigan that the Nominating Committee consist of the six men previously proposed. Motion seconded. Motion carried.

Moved by Mr. Thomas R. Camp that the Clerk cast one ballot for the 6 men as the Nominating Committee. Motion seconded and carried by voice vote. No negative votes.

Chairman Knowlton introduced Messrs. Wm. E. Stanley, Professor of Sanitary Engineering at M.I.T., and Warren J. Kaufman, Research Associate at M.I.T. to present a paper titled "Sewer Design Practice—Past, Present and Future".

Professor Stanley presented a 20-minute resume of current practice in computing sewage quantities for sewer design, including six lantern slides of illustrative charts and then introduced Mr. Warren J. Kaufman who discussed the theory and application of the Hydrogen method of determining storm water quantities for sewer design, with references to the use of this method by Mr. Hicks of Los Angeles.

At the invitation of Chairman Knowlton, Mr. Thomas R. Camp presented a lengthy review of the hydraulics of sewers referring back to the paper he presented on this subject some time ago before this Sanitary Section. There were discussions by Messrs. Cahaly, Brousseau and several others.

The meeting adjourned at 9:45 P.M.

WILLIAM E. STANLEY, *Clerk*

STRUCTURAL SECTION

OCTOBER 26, 1949, at Tremont Temple Social Hall: This was a joint meeting with the Surveying and Mapping Section. Chairman Hugh P. Duffill of that section presided, and introduced Mr. Nathan Rossman of New England Survey Service, Inc., who presented a paper on "Surveying and Construction of the Mystic River Bridge".

The paper covered the preliminary surveys, the staking out and control during construction (performed by parties of the construction contractors and checked independently by parties representing the consulting engineers) and included a general description of the construction, the latter being illustrated by slides.

The attendance was 105.

EDWARD C. KEANE, *Clerk*

NOVEMBER 16, 1949, at the Society rooms: Chairman Spencer introduced Mr. Francis J. Mardulier of the Dewey and Almy Chemical Company who presented an illustrated paper on the manufacture of portland cement.

The speaker first discussed the nature

of cement and gave some historical background, then discussed the reasons for the limitations on the various constituents imposed by the specifications for the various types. He followed with a description of the processes and machinery in a typical mill. A discussion period followed.

The attendance was 25.

EDWARD C. KEANE, *Clerk*

DECEMBER 14, 1949, at Chipman Hall, Tremont Temple. This was a joint meeting of the Boston Society of Civil Engineers and the Structural Section, B.S.C.E.

The speaker of the evening was Mr. John J. Jenkins, Jr., Chief Engineer, J. E. Greiner Company, Consulting Engineers of Baltimore, Maryland, who presented a paper entitled "Design Problems Attendant to the Mystic River Bridge". Mr. Jenkins treated the subject broadly, and included historical, legislative and financial background as well as a description of design procedures, construction methods, and the resulting structure. The talk was illustrated by lantern slides. A discussion period followed.

One hundred six members and guests attended.

EDWARD C. KEANE, *Clerk*

TRANSPORTATION SECTION

OCTOBER 19, 1949.—A Joint Meeting with the Main Society was held in Social Hall, Tremont Temple, Boston, Mass., and was called to order by President Harrison P. Eddy, Jr., at 7::00 P.M. The reading of the minutes of the preceding meeting was waived. The members were asked to rise and President Eddy announced the death of the following member:

Bertram D. Brewer, who was elected a member May 20, 1896, and who died August 8, 1949.

The meeting was then turned over to Mr. Hyland, Chairman of the Transportation Section. The reading of the minutes of the last meeting of the Transportation Section was waived. On motion of Ernest Spencer which was duly seconded, the Chair was authorized to appoint a nominating committee to present a slate of officers for presentation at the annual meeting to take place in February.

Mr. Hyland introduced the speaker of the evening, Mr. John D. M. Luttman-Johnson, Engineer with Fay, Spofford & Thorndike, whose subject was "Harbor Engineering, with Particular Reference to Development of Port Elizabeth, Union of South Africa". With the aid of lantern slides and prepared text Mr. Johnson described the design and construction of the breakwater, quays, marine railway, transit sheds, and other facilities of the artificial harbor at Port Elizabeth.

The attendance was fifty-two.

WILLIAM L. HYLAND, *Chairman*

HYDRAULICS SECTION

NOVEMBER 2, 1949. A meeting of the Hydraulics Section was held at the Society Rooms on this date, following a dinner at Patten's Restaurant.

The meeting was called to order at 7:30 P.M. by Chairman James F. Brittain. During a brief business session the minutes of the last meeting were read and accepted and the chairman was empowered to appoint the last three chairmen of the Section as a Nominating Committee to present a slate of officers at the annual meeting in February.

The speaker of the evening was Ralph S. Archibald of Camp, Dresser & McKee, Consulting Engineers. Mr. Archibald, who in 1948 was awarded the John R. Freeman Fellowship of the Boston Society of Civil Engineers, discussed his project which was research in "Radioactive Tracers in Flow Tests." His talk indicated that his investigations at Harvard and in the field have

shown the usefulness of radioactive tracers in the realm of hydraulics. The speaker demonstrated instruments used for measuring radioactivity.

Mr. Archibald's interesting talk was followed by discussion by Professor Harold A. Thomas. Further discussion by others emphasized the interest in the subject. The meeting adjourned at 9:15 P.M.

Seventy members and guests attended the meeting.

GARDNER K. WOOD, *Clerk*

SURVEYING AND MAPPING SECTION

OCTOBER 26, 1949. The ninth meeting of the Surveying and Mapping Section was held as a joint meeting with the Structural Section at the Society Rooms at 7:30 P.M.

Approximately one hundred and five members of both sections and guests were present.

A short business meeting was held and a nominating committee of Wilbur C. Nylander, Charles M. Anderson and C. Frederick Joy, Jr., was elected to present a list of officers at the Annual Meeting.

Chairman Hugh P. Duffill introduced the speaker of the evening, Mr. Nathan Rossman, of the New England Survey Service, Inc., who spoke on the surveying and construction of the Mystic River Bridge. Following the talk numerous slides were shown showing the various phases of construction after which a discussion period was held.

The meeting adjourned at 8:30 P.M.

JOHN H. LOWE, *Clerk*

APPLICATION FOR MEMBERSHIP

[January 1, 1950]

The By-Laws provided that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character of professional reputation and experience of the candidates which will assist the Board in its considerations. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

GEORGE C. CAPELLE, JR., Duxbury, Mass. (b. September 21, 1914, Newton, Mass.) Graduated from Dartmouth College with A.B. degree in 1936. Experience: 1936-1938, leadsman, recorder, instrumentman with U. S. Engineer Dept. on hydrographic surveying on Boston Harbor; 1938, April-September, Civil Engineer with J. S. Packard Dredging Company; October, 1938-April, 1939, transitman in survey party on highway and bridge surveys for Mass. Dept. Public Works Highway Division in Pittsfield District; May, 1939-August, 1939, transitman with Met. Dist. Water Supply Comm.; August, 1939-February, 1943, inspector of construction with U. S. Engineer Dept. on Flood Control dikes and stop-log structures in Hartford and East Hartford, Conn.; March, 1943 to January, 1944, Chief of survey party with U. S. Engineer Department on Airport construction at Camp Edwards, Mass.; January, 1944-March, 1945, Chief of survey party with U. S. Engineer Dept. on hydrographic surveying, Boston Harbor and vicinity; March, 1945-July,

1946, Ensign USNR, Civil Engineer Corp., Executive Officer on 30" hydraulic dredge, based at Guam; July, 1946 to present, Fay, Spofford & Thorndike, inspector of dredging and chief of survey party on layout and soundings for dredging. Office computations and drafting; carried on investigation and surveys for design of additions to sewer system in New Bedford. Resident Engineer on two sewer jobs in Walpole, Mass. At present an Assistant Resident Engineer on construction of additions to sewer system in New Bedford, Mass. Refers to J. A. Christenson, F. L. Heaney, A. W. Caird, F. L. Lincoln, N. Wentworth, Jr.

ROLF ELIASSEN, Winchester, Mass. (b. February 22, 1911, Brooklyn, N. Y.). B.S. in C.E., M.I.T. 1932; M.S. in C.E., 1933; Sc.D. in San. Eng. M.I.T., 1935; Experience: Design Engineer, The Chester Engineers, Pittsburgh, Pa.; 1935-1936, design of water and sewage treatment plants; 1936-1939, Sanitary engineer, The Dorr Company, Inc., on the design, construction and operation of water, sewage, and industrial waste treatment plants all over the U. S. while assigned to the New York, Chicago, Kansas City, Denver and Los Angeles offices; 1939-1940, Asst. Prof. of Civil Engineering, Illinois Institute of Technology, Chicago; 1940-1942, Associate Prof. of Sanitary Engineering, College of Engineering, New York University; 1942-1946, Capt., Major, and Lt. Col., Corps of Engineers, U. S. Army. In charge of sanitary engineering work for Second Service Command, New York, and Ninth Service Command, Salt Lake City; 1946-1949, Professor of Sanitary Engineering, College of Engineering, New York University and Consulting Engineer to the City of New York, the State of New York and the U. S. Atomic Energy Comm. At present present Prof. of Sanitary Engineering, in charge of Sanitary Engineering Division of Dept. of Civil and Sanitary Engineering, Massachusetts Institute of Technology. Refers to, H. P. Eddy, Jr., E. S. Chase, T. R. Camp, F. L. Flood.

NORMAN R. HAMILTON, Cambridge, Mass. (b. June 5, 1897, Cambridge, Mass.). 1914-1917, Mass. Institute of Technology, Industrial Physics, Class of 1918; 1917-1919, World War I—Captain, U. S. Cavalry (R.A.); 1919, Plant Engineering, Large Shipyard, Newport News, Va.; 1919-1920, Industrial Engineer, Soap Mfg., Mass.; 1925-1931, Planning, Surveys, Development, Location of New Industries, Norfolk, Va.; 1928 (part) Plant Location and Raw Materials investigations, Far East, Malaya, India; 1932-1933, Study of Law, Virginia; 1933-1934, Highway Roads, Street Construction, Project Engineer, U. S. Government, Langley Field Air Base, Va.; 1938-1942, General Civil Engineering, Assistant Utilities Engineer U. S. Engrs. (& QMC) U. S. Government, Fort Monroe, Virginia and Vicinity; 1941-1946, World War II, Lt. Colonel, Corps of Engineers (Capt., Major) Post Engineer, various and Middle Atlantic Division HQ Staff. At present with Fay, Spofford & Thorndike, Engineers, Boston, Mass., engineer of Contracts and Specifications. Member—American Society of Military Engineers, 1942 to date. Attorney at Law—Admitted to the Bar, Virginia, June 30, 1933. Refers to K. R. Garland, W. L. Hyland, F. L. Lincoln, H. J. Williams.

ALICE M. LICHWELL, West Roxbury, Mass. (b. March 25, 1926, Norwich, Conn.) .Two years Civil Engineering, Northwestern University; 2½ years Structural Engineering, Lincoln Technical Institute; 1 year Statistics, Boston University evenings. Experience: six months, Raytheon Mfg. Company, Electrical and Mechanical Drafting; one year Keystone Custodian Funds, Mathematician and Statistician; one and one-half years, Holtzer, Cabot

Electrical Company, statistician, production control, time & motion study, quality control engineer; one-half year, Jackson & Moreland, structural drafting. Past eight months Structural Design with Drummey-Duffill, Inc., Boston, Mass. Refers to C. O. Baird, H. P. Duffill, O. G. Julian, E. L. Spencer.

FRANCIS A. MELAUGH, Jamaica Plain, Mass. (b. January 20, 1916, Boston, Mass.). January, 1934, to February, 1939, architectural draftsman for Boston School Building Department, drafting on school building; February, 1939, to December, 1939, civil engineering aide for Metropolitan Sewerage Division, work as transitman giving line and grade for a shield driven tunnel; December, 1939, to March, 1940, with Silas Mason Construction Company, performed the duties of a groutman in a shield driven tunnel; April, 1940, to August, 1940, safety engineer for Liberty Mutual Insurance Company, inspected all types of construction from the safety standpoint and prepared reports; September, 1940 to July, 1941, Stone & Webster, Engineering Corp., designing draftsman. Drafted and designed steel and concrete structures; July, 1941, to February, 1946, U. S. Army, Corps of Engineers. Duties of company commander, engineering and operation officer for a construction group. Left Army with the rank of Major; May, 1946, to March, 1947, Junior Civil Engineer, Boston School Building Department, supervisor of construction in the department; March, 1947, to June, 1949, Senior Structural Engineer, Port of Boston Authority, in charge of the design and drafting section of the authority; June, 1949 to date, Chief Airport Engineer, Massachusetts Aeronautics Commission, in charge of the engineering section of the commission whose duties require the approving of all plans with specification for all municipal airports. Refers to E. S. Averell, H. P. Duffill, E. L. Spencer, F. N. Weaver.

ARTHUR H. MOSHER, JR., New Bedford, Mass. (b. May 3, 1917, New Bedford, Mass.). Attended Massachusetts Institute of Technology for 2 years and 3 months, left November, 1938. Experience: 1938-1940, City of New Bedford, Engineering Department as Grade II Civil Engineer on sewer construction; 1940-1945, on active duty U. S. Corps of Engineers as Master Sgt. Operations Chief with duties consisting of overlays, surveys and arrangement of forces for movement without mishap; 1945-1947, U. S. Rubber Company, Fisk Cord Mills, New Bedford as Cost Engineer to maintain methods of fabric production and install new methods where applicable; at present employed by Theodore Loranger & Sons and A. Piatelli Company, New Bedford, as Cost Engineer for Intercepting Sewer System have been maintaining cost system (unit) and investigation terrain and coordinating movements of contractor to keep a smooth operating organization. Refers to J. Christenson, F. L. Heaney, B. A. Lekesky, P. Howard.

Carney M. Terzian, Cambridge, Mass. (b. September 7, 1926, Cambridge, Mass.) Graduated from Northeastern University on January 31, 1948, with B.S. degree in Civil Engineering. Cooperative work at Boston Edison Company as a laboratory assistant from June to September, 1945. Thru 1946 worked at the Keystone Mfg. Company as an assistant to the plant engineer from February to September of that year. Upon graduation from Northeastern University have been employed as a Senior Engineering Aide with the Metropolitan District Commission, Construction Division from March 8, 1948 to date. Refers to S. M. Dore, C. J. Ginder, L. M. Gentleman, K. R. Kennison, R. W. Moir.

Transfer From Grade of Junior

JOHN J. CUSACK, Dorchester, Mass. (b. April 11, 1922, Boston, Mass.). Graduated from Northeastern University in 1947, Civil Engineering, B.S.C.E. September, 1943-December, 1943, U. S. Naval Academy, Eng. Midshipman School; December, 1943-May, 1944, Penn. State College, Diesel School; September, 1949, Northeastern University, Engineering Management. Experience: June, 1941-November, 1941, U. S. Army Engineer, Franklin Falls, N. H.; January, 1942-March, 1942 and June, 1942 to December, 1942, U. S. Army Engineers, Castle Island, Boston, Mass., worked as a transitman and rodman during the construction of the docks. December, 1942-June, 1946, and December, 1947-March, 1949, U. S. Navy, Engineering officer on a Landing Ship Tank. Detailer of reinforcing bars and steel joists; June, 1947-December, 1947, Donnelly Advertising Company, designed the trusses and anchorages to support roof signs; March, 1949-October, 1949, U. S. Army Engineers, P.2 structural engineer. Performed structural design on various government projects. Refers to J. H. Brown, J. Lavin, E. A. Gramstorff, E. L. Spencer.

JAMES M. ROSA,, Brighton, Mass. (b. August 11, 1921, Cambridge, Mass.). Entered Northeastern University September, 1940. Education was interrupted by war and served as a Navigator with AAF with rank of 1st Lt. Entered school again after service and graduated in June, 1947. Experience: with Metcalf & Eddy until February, 1949, when I entered business for myself under the Rosa & Todisco Construction Company where we are engaged in the construction of sewers, water mains, roads, etc. Refers to E. B. Cobb, F. L. Flood, E. A. Gramstorff, R. J. Rice.

Transfer from Grade of Student

HENRY J. BISHOP, New Bedford, Mass. (b. October 31, 1918, New Bedford, Mass.). Graduated from Northeastern University with B.S. degree in 1944. Experience: 1944-1945, structural engineer for National Advisory Committee on Aeronautics; 1945-1948, Turner Construction Company, as field engineer on lines and grades, assistant supt. and supt. of construction. Field office engineer on the new Telephone Exchange Building in the city of Boston. Holder of A-B-C Building License in the city of Boston; 1948-1949, construction superintendent for Theodore Loranger & Sons of New Bedford. Supervised the construction of radio station WBSM on Pipe's Island, New Bedford, Mass. Presently employed by Theodore Loranger & Sons and the A. Piatelli Company as construction Supt. on two pumping stations in conjunction with the Additions to the Intercepting Sewer System for the City of New Bedford, Mass. Refers to E. A. Gramstorff, F. L. Heaney, J. Christenson, C. S. Ell.

LEON P. PIATELLI, Hyde Park, Mass. (b. June 25, 1920, Boston, Mass.). Graduated from Northeastern University with B.S. degree in 1943. Experience: 1944-1946, 59th Naval Construction Battallion; 1941-1943, Turner Construction Company, field engineer (as part of co-op plan at University); 1947-1949, engineering, estimating and job supt. for A. Piatelli Company, contractors constructing sewer, water & electric lines at cities of Haverhill, Sagamore, Newton and Worcester. Presently employed as assistant supt. of construction and Partner of T. Loranger & Sons & A. Piatelli Company, doing Interceptor Sewers for the City of New Bedford. Refers to C. O. Baird, J. Christenson, F. L. Heaney, E. A. Gramstorff.

ADDITIONS

Members

M. Gordon MacInnes, 119 Kenrick Street, Brighton, Mass.

Clair N. Sawyer, 451 School Street, Belmont, Mass.

Thomas J. Rouner, Sandy Pond, Lincoln, Mass.

Juniors

Harold J. Publicover, 36 Hazel Street, Milton, Mass.

DEATHS

LUTHER H. BATEMAN, October 26, 1949
BERTRAM D. BREWER, August 8, 1949
ALMON L. FALES, August 19, 1949
CHARLES E. NICHOLS, July 3, 1949
GEORGE G. SHEDD, August 31, 1949

PROFESSIONAL SERVICES AND ADVERTISEMENTS

The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

Volume 37 JANUARY, 1950 Number 1

BOSTON SOCIETY OF CIVIL ENGINEERS

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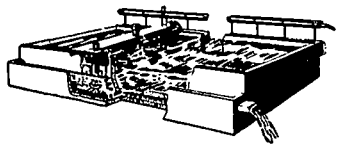

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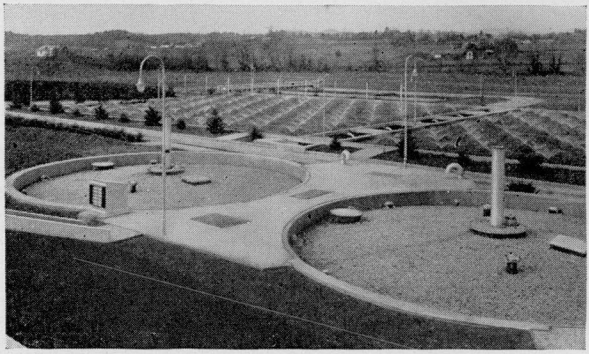

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